AMERICAN SEWERAGE PRACTICE

VOLUME I

DESIGN OF SEWERS

The quality of the materials used in the manufacture of this book is governed by continued postwar shortages.

AMERICAN SEWERAGE PRACTICE

VOLUME I
DESIGN OF SEWERS

BY
LEONARD METCALF
AND
HARRISON P. EDDY

Second Edition
Ninth Impression

McGRAW-HILL BOOK COMPANY, Inc.
NEW YORK AND LONDON
1928

PRINTED IN THE UNITED STATES OF AMERICA

THE MAPLE PRESS COMPANY, YORK, PA.

PREFACE TO THE SECOND EDITION

The death of Mr. Metcalf in January, 1926, left upon the surviving author the responsibility for a revised edition of this work. It is fitting that in this new edition acknowledgment should be made of the energy and enthusiasm devoted by Mr. Metcalf to the original preparation of the book. But for his willingness to devote a large amount of time to the accumulation and digestion of the material required, it is doubtful if the book would have ever appeared. The commendation which it has received was a source of much gratification to him.

In the fourteen years which have elapsed since the publication of the first edition, American practice in sewer design has attained a greater degree of uniformity, considerable advance has been made in the methods of application of principles, and there has been material increase in the amount of pertinent information available. A revised edition has, therefore, become desirable.

In the new edition there has been a considerable rearrangement of material; some chapters have been divided and others consolidated, and many sections have largely been rewritten. It is believed that the book has been made more nearly complete as a work of reference.

The work of revision has extended over several years. A material portion of it had been accomplished before the death of Mr. Metcalf and had the benefit of his criticism. Important parts of the work have been done by Charles W. Sherman and Frank A. Marston, partners in the firm of Metcalf and Eddy, and by John W. Raymond and Herman G. Dresser of the staff, while other partners and employees have helped to a lesser degree. Professor R. G. Tyler, of the Massachusetts Institute of Technology, has also shared in bringing some chapters down to date, and in editing the whole volume.

HARRISON P. EDDY.

STATLER BUILDING,
BOSTON, MASS.
October 26, 1928.

PREFACE TO THE FIRST EDITION

About three years ago, the authors undertook the preparation of a book bringing together in a form convenient for ready reference the more important principles of theory and rules of practice in the design and operation of sewerage works, using this term in its broadest sense. It was found, however, that to make these fundamental data, tables, diagrams, and rules of the greatest service, it was desirable to explain them in some detail, for such explanations can be found only scattered through many special treatises, transactions of technical societies, engineering journals, and reports. In some cases, it developed, to the authors' surprise, that nothing really definite had ever been published concerning many important features of sewerage practice. In other cases, the practice of different engineers, being based upon their individual experiences, varied considerably. These conditions led the authors to broaden the scope of the work and to devote considerable space to topics upon which little had been written, in order that the reader might find all of the information which it was reasonable to expect in a comprehensive review of a subject of such scope as sewerage practice. It thus became necessary to present the subject in three volumes, the first dealing with the Design of Sewerage Systems, the second with their Construction, and the third with the Design of Works for the Treatment and Disposal of Sewage.

As the various chapters of this volume have developed, much interest has been shown in the work by different engineers, both at home and abroad, who have supplied many helpful suggestions, valuable statements of their views upon subjects where experience furnishes a guide often more helpful than theory, and drawings of special structures to illustrate their standard practice in design. To these engineers hearty thanks are given for their cordial assistance in the authors' attempt to outline standard practice and sound principles of design.

The engineering journals have proved of valuable help, particularly in affording examples of practice and for their records of the development of present-day methods; and many manufacturers have been most courteous in supplying drawings, photographs, and specific information.

In the preparation of certain chapters of this volume, special aid has been obtained from "The Theory of Loads on Pipes in Ditches," by Prof. Anson Marston and A. O. Anderson (Iowa State College of Agriculture and Mechanic Arts); "A Treatise on Concrete, Plain and Reinforced," by Frederick W. Taylor and Sanford E. Thompson (John

Wiley & Sons, Inc.); "Principles of Reinforced Concrete Construction," by Profs. F. E. Turneaure and E. R. Maurer (John Wiley & Sons, Inc.); "A Treatise on Hydraulics," by Prof. Hector J. Hughes and Arthur T. Safford (copyright, 1911, The Macmillan Company); "American Civil Engineer's Pocket Book," edited by Mansfield Merriman (John Wiley & Sons, Inc.); and "A Treatise on Masonry Construction," by Prof. Ira O. Baker (John Wiley & Sons, Inc.). While acknowledgment has been made in the several chapters for this help, more specific thanks are here given for the generous permission to make such free use of these valuable contributions to engineering literature. The authors have also drawn upon the late August Frühling's valuable "Entwässerung der Städte," published by Wilhelm Engelmann.

The authors are under obligations to their junior partners, Charles W. Sherman, William T. Barnes, and Almon L. Fales, and to their office staff, particularly William L. Butcher and Frank A. Marston, for valuable assistance in the preparation of this book, and to John M. Goodell, for many years editor-in-chief of *Engineering Record*, whose constructive criticism and assistance in the preparation of the manuscript have been most helpful. To the publishers, the McGraw-Hill Book Company, Inc., whose work speaks for itself, thanks are also given.

Whatever its merits or demerits, the book is at least a monument to cooperative effort and goodwill among civil engineers.

The preparation of this book has demanded an amount of time and effort far in excess of that anticipated when the work was undertaken. The authors have carried it through, however, because of their experience of the practical value of such information as is given in many chapters herein. As problems have arisen in their work, reference has been made to the book for the help required, and if anything was found lacking it was supplied. This practical test has resulted in the repeated revision of large portions of many of the chapters. The book is published, therefore, with the belief that it is a "practical" book, but as the test of service in one office is not a thorough test of a book on American Sewerage Practice, considered comprehensively, the authors will be glad to receive from the reader any suggested additions, changes, or modifications which will make the book more helpful and to have any errors of statement or computation called to their attention.

LEONARD METCALF.
HARRISON P. EDDY.

14 BEACON STREET,
BOSTON, MASS.
May 14, 1914.

CONTENTS

CHAPTER XIV

CHAPTER XV

CHAPTER XVI

CHAPTER XVII

CHAPTER XVIII

NOMENCLATURE

The following list includes the principal symbols used in this book, more particularly in the sections relating to hydraulics. Two or more meanings are given for some symbols, but there should be no confusion in their use, as the kind of formula in which they appear or the accompanying text will always show which meaning is applicable. Other uses for the same symbols, as well as some symbols not listed below, will be found in some portions of the book; these are explained or defined as they occur.

Except as otherwise noted, the units are the *foot, second,* and *pound.*

A = area of cross-section; drainage area in acres
c = coefficient of discharge
C = coefficient in the Chezy formula
C_h = coefficient in the Hazen-Williams formula
d = diameter (in inches)
d' = depth
D = diameter (in feet)
f = coefficient of friction
g = constant of gravitation = 32.16+
h = head or height of water above given datum
h_e = height of any point above given datum
h_f = friction head loss
h_p = pressure head
h_v = velocity head = $v^2/2g$
h_1 = head at upper of several points
h_2 = head at lower of two points
H = total head; head on weir corrected for velocity of approach
i = intensity of rainfall (in inches per hour)
l = length of weir crest
L = length
m = coefficient of roughness in Bazin formula
M = drainage area in square miles
n = coefficient of roughness in Kutter formula
N = number of end contractions of weir
P = wetted perimeter: intensity of pressure (per square inch)
Q = rate of discharge
r = radius
R = hydraulic radius
S = slope (expressed decimally)
t = time: duration of rainfall (in minutes)
T = temperature (in degrees Fahrenheit)
v = average or mean velocity
w = unit weight (of water = 62.5 lb. approx.)
Z = height of weir crest

AMERICAN SEWERAGE PRACTICE

INTRODUCTION: THE LESSONS TAUGHT BY EARLY SEWERAGE WORKS

American sewerage practice is noteworthy among the branches of engineering for the preponderating influence of experience, rather than experiment, upon the development of many of its features, apart from those concerned with the treatment of sewage. Even the actual capacity of sewers, something that gagings can determine, is far less clearly known today than is the capacity of water mains, while the cross-sections of masonry sewers and the forms of accessory structures employed under similar conditions in different cities vary widely. There has been, however, a rather decided tendency toward greater uniformity in design in recent years, keeping abreast with the growing popular recognition of the financial and sanitary importance of good sewerage and the passing of the feeling that it was a bit indelicate to speak in public of anything so unclean as sewage. Sewerage systems, being out of sight, were out of mind, except to the few intrusted with their construction and maintenance, and even today the lack of anything above ground to show to the taxpayer makes sewerage work in a city one of its least appreciated activities. The strong feeling that good public health is a valuable municipal asset and depends, to a large extent, upon good sewerage has been a leading cause of the willingness of taxpayers recently to embark on expensive sewerage undertakings.

The value of arousing public feeling toward sewerage in this way is the main lesson which its history teaches. Until it became a strong influence, sewerage work was concerned mainly with surface drainage and the abatement of nuisances. The first record of a sewer which Curt Merckel, the antiquarian of engineering, has been able to find is on an old Babylonian seal cylinder. Layard's explorations revealed arched sewers in Nineveh and Babylon dating from the seventh century before Christ. Schick and Warren have unearthed considerable information about the sewers of Jerusalem; the works of this class in Grecian cities are fairly well known, and the great underground drains of Rome have repeatedly been described. It is known, however, that these channels

and conduits were not used to any substantial extent by means of direct connections to them from the houses, for the requirements of public health were little recognized then and compulsory sanitation would have been considered an invasion of the rights of the individual. Livy states that the Roman building regulations only stipulated that the house connections were to be made at the cost of the property owners. Public latrines were doubtless used by most of the people and it is probable that the gutters were the chief receptacle of the ordure of the city, which was washed thence into the sewers. These must have been extremely offensive when not flushed, for otherwise the regular delivery of water for the purpose of cleaning them would not have been so emphasized in the following notes by Frontinus, a water commissioner of the city whose valuable notes of engineering work have been edited by Clemens Herschel:

> I desire that nobody shall conduct away any excess water without having received my permission or that of my representatives, for it is necessary that a part of the supply flowing from the water castles shall be utilized not only for cleaning our city but also for flushing the sewers.

It is astonishing to reflect that from the day of Frontinus (the first century A.D.) to that of W. Lindley,[1] there was no marked progress in sewerage. The renaissance began in Hamburg, where a severe conflagration destroyed the old part of the city in 1842. The portion ruined was the oldest section and it was decided to rebuild it according to modern ideas of convenience. This work was intrusted to Lindley, who carried it out in a way that aroused warm praise among engineers of a somewhat later period, when the test of service had placed the seal of approval on the plans.

For instance, E. S. Chesbrough, Moses Lane, and Dr. C. F. Folsom reported to the authorities of Boston in 1876 that Hamburg

> . . . was the first city which had a complete systematic sewerage system throughout, according to modern ideas. How far that was in advance of the rest of the world, in 1843, when the work was undertaken, may be inferred from the fact that there are no real advances in new principles from that time up to the present day. The rain-water spouts were all untrapped to serve as ventilators to the sewers; the street gullies were also without traps, and there were gratings for ventilation opening into the streets. It is very rare that any of the latter are sources of complaint, inasmuch as the sewers are kept so clean that there are seldom any foul-smelling gases. The great feature in Hamburg, however, is the weekly flushing at low tide by letting the waters of the Binnen-Alster flow through the sewers with great force.

[1] Lindley was one of the leading English engineers of his day, Rawlinson being his only rival at the head of the sanitary branch of his profession. He became thoroughly identified with German work, however, first at Hamburg and later at Frankfort.

Twenty-five years after the sewers were completed they were found by a committee of experts to be clean and almost without odor.

The sewerage of Hamburg, while indicative of an awakened public recognition of the need of improvement in such works, was hardly the result of any real appreciation of the value of sanitation but was, rather, the result of business shrewdness in taking advantage of exceptional local conditions to plan streets and sewers to answer in the best way the recognized needs of the community and the topographical conditions. The history of the progress of sanitation in London probably affords a more typical picture of what took place quite generally about the middle of the nineteenth century in the largest cities of Great Britain and the United States.

A statute was passed in 1531 in Henry VIII's reign[1] and amended in that of William and Mary which afforded the legal basis of all sanitary works of sewerage well into the nineteenth century. For a period of about 300 years, while London outgrew the narrow limits of the city proper and its adjacent parishes and became a great metropolis, the center of the world's commerce, sanitation was as little considered as magnetism or the utilization of steam for power purposes. The city was better off than most of the metropolitan district, for it had Commissioners of Sewers elected annually by the Common Council from its members. They had power over all conditions relating to public health and comfort and had authority to appoint a medical officer of health. But the city was only a small part of the metropolitan area, 720 out of 75,000 acres in 1855, with only 128,000 out of a total population of 2,500,000, and less than 15,000 out of a total of 300,000 houses. Outside of the city, the methods of local government were chaotic; in some of the parishes, surveyors of highways were appointed to do very restricted engineering work and in eight there were Commissioners of Sewers, apparently having powers modeled after those of London but less extensive.

This lack of central authority rendered a systematic study and execution of sewerage works impossible. As late as 1845 there was no survey of the metropolis adequate as a basis for planning sewers. The sewers in adjoining parishes were on different elevations so that a junction of them was impracticable.

> Some of the sewers were higher than the cesspools which they were supposed to drain, while others had been so constructed that to be of any use the sewage would have had to flow uphill. Large sewers were made to discharge into small sewers.[2]

The first engineer to make a comprehensive study of metropolitan

[1] Statutes relating to local drainage problems had been passed in the reigns of Henry III, Henry VI, and Henry VII.

[2] Jepson, "Sanitary Evolution of London."

sewerage needs in an official capacity, John Phillips, gave this testimony[1] of the condition of London basements and cellars in 1847:

There are hundreds, I may say thousands, of houses in this metropolis which have no drainage whatever, and the greater part of them have stinking, overflowing cesspools. And there are also hundreds of streets, courts, and alleys that have no sewers; and how the drainage and filth are cleaned away and how the miserable inhabitants live in such places, it is hard to tell.

In pursuance of my duties from time to time, I have visited very many places where filth was lying scattered about the rooms, vaults, cellars, areas, and yards, so thick and so deep that it was hardly possible to move for it. I have also seen in such places human beings living and sleeping in sunk rooms with filth from overflowing cesspools exuding through and running down the walls and over the floors . . . The effects of the effluvia, stench, and poisonous gases constantly evolving from these foul accumulations were apparent in the haggard, wan, and swarthy countenances and enfeebled limbs of the poor creatures whom I found residing over and amongst these dens of pollution and wretchedness.

One of the main reasons for the backward condition of the sewerage system in London for many years was the absence of authority to compel landlords to connect their houses with sewers, so that even the residences of the wealthiest members of the nobility were likely to be located over one or more cesspools, some of which were of enormous size. Even in Westminster, very little use was made of the sewers in some of the streets. "So long as the owners get the rent, they do not care about the drainage," the Commissioners of Sewers reported in 1845. It was not until two years later that the first act was passed making it compulsory to connect houses with sewers.

In 1847, scared by an outbreak of cholera in India, which had begun to work westward, a royal commission was appointed to inquire into sanitary improvements for London. This body reported that the sewerage of the entire metropolitan district should be handled by a single board, and in 1848 Parliament followed this advice and created the Metropolitan Commission of Sewers. That body and its successors in the office unfortunately failed to measure up to their opportunities; they produced reports showing clearly the need of extensive sewerage

[1] Lest this picture be considered too fanciful, a statement published in 1852 by the General Board of Health may be quoted here:

"During the first labors of the General Board of Health much illness prevailed among the clerks, until on one occasion foul smells arising more severely than had before been noticed, the state of the foundations was examined, when it was discovered that there were two very large cesspools immediately beneath the Board's offices. This is the description of houses of which it is generally reported by house agents and others that they are well drained and in good condition; but it may be advised that it is absolutely unsafe to take any house without a thorough examination of the site beneath it, nor where any cases of fever, typhoid or gastric have occurred amongst persons living in the lower offices of a house, is it safe for those who value their own health to remain in the premises without such an examination, nor until the cesspools are removed."

works and other sanitary improvements, built the Victoria sewer at great expense, which fell into ruins not many years later, but did little more. In the summer of 1848, cholera was discovered in London and before the winter was over it claimed 468 victims. It broke out again in the spring of 1849 and before it ended, about 14,600 deaths were recorded, as against 6,729 in London in the 1832–1833 epidemic.

In 1852 cholera again appeared and in 1853 it slowly gained a foothold. In 1854 it ran its terrible course, claiming a mortality of 10,675 in the last half of that year. The connection between a contaminated water supply and the rapid spread of the disease was clearly shown, but it was also apparent that the filthy living conditions in most houses, due to the absence of effective sewerage, was a great hindrance in combatting the scourge. In 1855 Parliament passed an act "for the better local management of the metropolis"; this laid the basis for the sanitation of London and provided for the Metropolitan Board of Works which soon after undertook an adequate sewerage system.

In this connection a brief mention of some of the features in the early development of the London sewers will be of value as showing, by contrast, the importance of the progress in sewerage in recent years.

In 1849, in answer to an advertisement, the Metropolitan Commission of Sewers received 116 different schemes for abating the nuisance due to sewage in the Thames; none was approved for execution. Plans for intercepting the sewage and conducting it to outlets below the city had been suggested many years before. Apparently, it was not until 1852, when J. W. Bazalgette became chief engineer of the Commission, that any beginning was made in formulating policies, although at least two engineers of high standing connected with the local works of certain subdivisions of the metropolitan district had been making valuable studies. Bazalgette seems to have been possessed of the executive ability previously lacking; he tentatively developed plans for interception and then worked them out in detail in collaboration with W. Haywood, the unusually gifted and highly respected engineer of the City Commissioners of Sewers, who had a thorough local engineering experience and was responsible for many of the basic assumptions upon which the plans were prepared. But no action was taken on these plans until the Metropolitan Board of Works appointed Bazalgette its engineer and he had been compelled to uphold them against lay and engineering criticism for several years. The works were not actually undertaken until 1859.

The old sewers were frequently the covered channels of brooks. The oldest was Ludgate Hill sewer, of unknown age, but built prior to Fleet Street sewer, which was constructed in 1668 and was an open channel for many years. This sewer, formerly known as the River of Wells or the Old Bourne (now called Holborn), was fed by several springs,

and was originally a navigable waterway from which people were supplied with water. It was not covered until 1732. What eventually became Ranelagh sewer was a brook rising in a spring at Tye Bourne and even as late as 1730 it furnished water for the Serpentine, the famous pond in Hyde Park. In 1855 the total length of these old sewers was 1,146 miles. Up to 1815 it was contrary to law to discharge sewage or other offensive matter into the sewers; cesspools[1] were regarded as the proper receptacles for house drainage and sewers as the legitimate channels for carrying off surface water only. The cesspools were cleaned by private contractors at the expense of the property owners, and consequently the frequency of the cleaning depended on the callousness of the owner or tenant to complaint and nuisance. Concerning the removal, the General Board of Health reported in 1852 as follows:

It appears that the quantity of cesspool refuse, including ordure and other animal and vegetable matter, is from 1 to 2 cu. yd. per house per annum; and the cost of its removal in London (including openings and making good the cesspool, and cartage out of town) was stated by contractors, and proved, upon a house-to-house inquiry, to be on an average, about 20s. per house. When cheap cesspools are made, from which percolation is not prevented, the injury to the foundations of the houses would more than make up the difference. In many country towns, where night soil is kept in shallow uncovered pits (called midden holes) the cost of emptying is less than where deep cesspools are used, but although the emanations, as being more diluted, may be less noxious than those arising from covered cesspools, the sight of the exposed ordure is offensive and degrading, and the open midden steads are, in other respects, serious nuisances.

It has been mentioned that the pollution of the Thames was a cause of public protest in the middle of the last century; it was aggravated by the manner in which the sewers discharged their contents. Bazalgette's description of it is worth quoting as explaining a feature of outfall sewer design which is sometimes overlooked at the present time:

According to the system which it was sought to improve, the London main sewers fell into the valley of the Thames, and most of them, passing under the low grounds on the margin of the river before they reached it, discharged their contents into that river at or about the level, and at the time only, of low water. As the tide rose, it closed the outlets and ponded back the sewage flowing from the high grounds; this accumulated in the low-lying portions of the sewers, where it remained stagnant in many cases for 18 out of every 24 hours. During that period, the heavier ingredients were

[1] "What are termed dry wells in the United States differ from the London cesspools in this particular: they consist of excavated pits in the subsoil, sustained by pervious masonry linings, and are not intended to be cleaned out until the surrounding earth fails to absorb their contents; while the London cesspools were constructed with impervious masonry linings and were designed to be cleaned out at proper intervals." (Report by Charles Hermany on Memphis sewerage, 1868.)

deposited, and from day to day accumulated in the sewers; besides which, in times of heavy and long-continued rains, and more particularly when these occurred at the time of high water in the river, the closed sewers were unable to store the increased volume of sewage, which then rose through the house drains and flooded the basements of the houses. The effect upon the Thames of thus discharging the sewage into it at the time of low water, was most injurious, because not only was it carried by the rising tide up the river, to be brought back to London by the following ebb tide, there to mix with each day's fresh supply—the progress of many days' accumulation toward the sea being almost imperceptible—but the volume of the pure water in the river, being at that time at its minimum, rendered it quite incapable of diluting and disinfecting such vast masses of sewage.

In designing the great intercepting and outfall sewers to remedy this condition, Bazalgette adopted a mean velocity of 2.2 ft. per second as adequate to prevent silting in a main sewer running half full, "more especially when the contents have been previously passed through a pumping station." The computation of the house sewage was based on an average density of population of 30,000 persons per square mile except in the outlying districts, where it was assumed at 20,000. The sewage was estimated at the assumed water consumption, 5 cu. ft. per capita daily.

This quantity varies but little from the water supply with which a given population is provided; for that portion which is absorbed and evaporated is compensated for by the dry-weather underground leakage into the sewers.

One-half of this sewage was assumed to flow off within 6 hours. The storm-water runoff,[1] for which provision was made, was a rainfall at

[1] "There are, in almost every year, exceptional cases of heavy and violent rain storms, and these have measured 1 in., and sometimes even 2 in., in 1 hour. A quantity equal to $\frac{1}{100}$ in. of rain in 1 hour, or $\frac{1}{4}$ in. in 24 hours, running into the sewers, would occupy as much space as the maximum prospective flow of sewage to be provided for; so that, if that quantity of rain were included in the intercepting sewers, they would, during the 6 hours of maximum flow, be filled with an equal volume of sewage, and during the remaining 18 hours additional space would be reserved for a larger quantity of rain. Taking this circumstance into consideration, and allowing for the abstraction due to evaporation and absorption, it is probable that if the sewers were made capable of carrying off a volume equal to a rainfall of $\frac{1}{4}$ in. per day, during the 6 hours of the maximum flow, there would not be more than 12 days in a year on which the sewers would be overcharged, and then only for short periods during such days." Bazalgette, *Proc. Inst. C. E.*, **24**, 292.

"The total sewage and rainfall provided for was 394,000,000 imp. gal. per day. The discharging capacity of the sewers was, however, made larger than this amount, as it is a well-known fact that, owing to the fluctuating flow of sewage at different hours of the day, about one-half of the total quantity flows off in 6 hours, and as figures in the above tables [108,000,000 imp. gal. of sewage and 286,000,000 imp. gal. of rain water.—Metcalf and Eddy.] give the flow of sewage spread over the whole 24 hours, provision had to be made and was made for practically double the amount of sewage given in these tables . . . This provision for discharging excessive rainfall into the Thames by means of the old sewers could not be satisfactory at all times, as it has already been pointed out that these old sewers were blocked by the tide for a considerable time before and after high water, and, therefore, the rainfall could only reach the Thames at some time on each side of low water, unless in any case the old sewers were capable of being put under such a pressure as would overcome the opposing pressure of the tidal head." Maurice Fitzmaurice, "Main Drainage of London."

the rate of 1/4 in. per day received during the 6 hours of maximum sewage flow, with overflows to discharge the excess due to larger amounts through some of the old sewers directly into the river.

It is not surprising, in the light of present information summarized in one of the following chapters, that these estimates proved too low and flooding took place in low-lying districts. As for the average minimum velocity selected, it was higher than that recommended by some contemporary engineers. Wicksteed had reported experiments showing that a bottom velocity of 16 in. per second would move heavy pieces of brick and stone, and a velocity of 21 3/4 in. would move iron borings and heavy slag. John Phillips advocated a velocity of 2 1/2 ft. per second. Professor Robison said, in his "Theory of Rivers," that a bottom velocity of 3 in. per second will take up fine clay such as potters use, 6 in. will lift fine sand, 8 in. will lift sand as coarse as linseed, 12 in. will sweep along fine gravel, 24 in. will roll along 1-in. pebbles, and 36 in. will move angular stones of the size of an egg. These statements of the state of knowledge in 1850 show a tendency to underestimate requisite velocities to prevent silting, and, taken in connection with the underestimates of runoff and their unpleasant consequences, illustrate the great desirability of adequate experiments to ascertain unknown facts essential for successful design, before spending great sums on construction.

The questionable character of the information available for design was recognized by a number of engineers, as the following remarks[1] by Sir Robert Rawlinson indicate clearly:

> To talk of a formula for main sewers, devised and drawn up from any one set of experiments, would only tend to mislead young engineers. There were no two places which required precisely the same treatment . . . The proper mode of proceeding was: before attempting to fix the dimensions of main sewers, to take the area to be operated upon as it existed; to consider what nature had previously done with that area; then to consider the special duties which the sewers had to perform, and apportion them to the water supply and to the probable increase of the population; and if the dimensions adopted were calculated for passing off three times or four times that volume, the engineer would not be far astray in this calculation.[2]

Prior to Haywood and Bazalgette's work on the London intercepting sewers, Phillips and Roe were prominently before the public as sewerage experts and among English-speaking engineers Roe's Table[3] was used

[1] It is evident that Sir Robert was speaking of sewers for house drainage only. He was the leader in the development of modern sewerage practice and exercised a great influence over the engineers and the public of his day.

[2] *Proc. Inst. C. E.*, **24**, 317.

[3] Roe's Table was not accepted by some contemporary London engineers, and in 1865, W. Haywood, engineer of the City, who remained for half a century a leading authority on English municipal engineering, stated at a meeting of the Institution of Civil Engineers that there were no reliable gagings of London sewers in existence and that he had never been

for many years in selecting the sizes of sewers. It was acknowledged to be entirely empirical and was said to be based on Roe's observations in the Holborn and Finsbury divisions of the London sewers during more than 20 years. It gave the areas which could be drained by sewers of various sizes and on various slopes, as indicated by that experience.

It should be said here that sewerage progress elsewhere in England was apparently less opposed than in London. In 1848, Parliament passed a sanitary code applying to all parts of England and Wales, except London, and in 1855 it enacted a nuisance removal law for all England; these laws were the basis of the subsequent sanitary progress outside the metropolis for many years. It will be observed, however, that the development of sewerage undertakings in that country was a direct result of the awakening of the people by a succession of epidemics of cholera, for progress did not begin until that disease had twice terrorized the country within a short period.

The present sewerage system of Paris, like that of London, was inaugurated as a result of a cholera epidemic. The system is unique in some ways, although in its early days the Parisian sewers were doubtless little different from the conduits enclosing old brooks or receiving storm water which were constructed in many large cities. The Menilmontant sewer, mentioned in a record of 1412, was of this type, and remained uncovered until about 1750. It intercepted the drainage of the streets on the northern slope of the city's area lying on the right bank of the Seine and was called the "great drain" (*grand égout* or *égout de Ceinture*). The part of the city on the left bank of the river was drained by open gutters leading down the centers of the streets to the river.

The first attempt to study the sewerage needs of the city comprehensively apparently was made in 1808, when there were 14½ miles of drains with about 40 outlets into the river, and during the next 24 years about 10½ miles more of drains were constructed. In 1832 the ravages of cholera awakened the authorities to a partial realization of the city's insanitary condition. The following year a topographical survey was made and, with the aid of the maps based upon it, five systems or divisions of sewerage were planned, based on topographical features of the territory rather than on the administrative boundaries of parishes, which caused so much delay in the development of rational drainage at London and have been harmful in the United States. Many of the low-lying streets along the river were raised at this time above the level of any known flood, which indicates that the work was regarded as drainage rather than house sewerage. The regulation of the streets was

---

able to obtain any accurate information regarding such work from either Phillips or Roe. He stated that he had been forced to make extensive gagings in consequence, and these showed that about half the sewage coming daily from the 11 square miles tributary to the gaging stations passed off between 9 A. M. and 5 P. M.

attended in some cases by the reconstruction or entire abandonment of the old sewers in them. One of the most interesting features of the work was the change in the cross-section of the streets from concave to convex, for reasons explained by H. B. Hederstedt:

> With regard to the conversion of the concave surfaces of roads into convex, it may be shown to have formed an important part of the drainage system. Against hollow roads, there were always complaints. The old plan constantly cut up the roadway with cross-channels or gutters. Another object had been considered, however, in making the change, the certainty of freeing the roads more readily from rainfall. In the concave roads, iron gratings were set on the top of small working shafts, built on the crown of the drain arch. These iron gratings frequently became clogged and the passage of the water was impeded to such an extent that raised planks were occasionally used to enable foot passengers to cross the road, the vehicles meanwhile being compelled to travel through a sea of mud. The old roadway had, in many places, to be lifted to obtain sufficient headway for the minimum-sized drains; the value of the convex roads, as affording an extra height, is therefore obvious.[1]

The new sewers built in Paris from 1833 onward were made 6 ft. or more high wherever possible, in the belief that the workmen employed in cleaning them would discharge their duties more efficiently if they could labor without being forced to take unnatural positions.[2] Toward 1848 the little sewers were given a minimum height of 5.5 to 5.9 ft. without exception, and a width of 2.3 to 2.6 ft. at the springing line of the arch, the width at the invert being a trifle less. Humblot says, "These dimensions are too scanty; for getting about easily at least 2 m. height and 1 m. width are needed." When it became necessary later to enlarge some of these small sections to receive water mains, the top was widened out on one side (sometimes on both sides) while the lower part was left narrow, thus producing those sections shaped something like the letter P which have been the subject of strange comments from persons unfamiliar with their origin.

Although there has been a great deal of criticism of the large Parisian sections it has generally failed to take into account that the sewers of that city have been built with a view to removing street refuse as well as sewage and rain water. There are no catchbasins on these great

[1] *Proc. Inst. C. E.*, **24**, 262.

[2] "One reason for making the smallest class of public sewers in Paris so much larger than they are in every other city is the practice which, till within 10 years, existed only there, of placing the water mains in them." (E. S. Chesbrough, 1856.) In commenting on the location of water mains in the sewers, Humblot stated in a report, in 1886, that the flat portions of Paris were largely on filled ground and the hills were undermined by old quarries, so that leaks in mains laid in earth would rarely be detected, and it was particularly desirable to keep the mains exposed so that their condition could be observed constantly. Telegraph and telephone lines and pneumatic tubes for transporting mail were placed in the sewers on account of the facility of installation and maintenance. Gas mains were also placed in a few of the sewers until explosions led to the abandonment of this practice.

drains, so that everything entering the inlets and not caught in the little baskets suspended in some of them, passes directly into the sewers. The streets are cleaned largely by washing them with hose streams. The street litter is flushed into the sewers and is swept down the latter by storm water and by the sewer-cleaning gangs into the larger sewers or collectors. Some of this sludge is removed through manholes but most of it is flushed through the collectors or intercepting sewers by the *bateaux vannes* and *wagons vannes*. These are boats or cars provided with wings reaching nearly to the walls of the channels. The wings dam up the sewage somewhat and it escapes around their edges with a higher velocity than that of the ordinary current. In this way the sludge is stirred up and carried along ahead of these cleaning devices. Other means of cleaning are also employed, but it is unnecessary to describe them here; the reader interested in the subject will find the whole field of the design, construction and management of the Paris sewer system described in Hervieu's "Traité Pratique de la Construction des Égouts" (Paris, 1897). A large part of the sludge is forced along into large chambers on the banks of the river, where it is discharged through chutes into barges which remove it to various places of disposal.

The chief feature of this work inaugurated in 1833 was its recognition of the principle of interception. Longitudinal drains of large section were laid out parallel to the river and only three of the forty old mouths of independent sewers were left in service, the remaining systems being made to discharge into the intercepters. The rain water falling on the roofs was taken at first through leaders to the gutters, but later was diverted in some cases to the large "house drains," with sections big enough for a man to walk through, connecting the houses with the sewers but used only for delivering waste water and not for excrementitious matter. The latter was discharged for many years into cesspools, one frequently answering for an entire block of houses.

The Parisians committed the fatal mistake, about 1820, of insisting by ordinance on cesspool construction. It was recorded that the whole subsoil of Paris was on the point of becoming putrid with cesspit matter, and that the ordinance was passed in consequence. By it all cesspits, as matters of private construction, were abolished, and the construction of cesspools on a gigantic scale was undertaken or directed by the municipality, and all persons thereafter building houses were obliged to construct "hermetically sealed cesspools" after a municipal or royal plan which had been devised by the government engineers of France. Into those cesspools effete matter from water closets, grease and washings from the sinks, and such refuse was to be discharged.[1]

The cesspools finally became so offensive that the nostrils of the Parisians were plagued and a new system of sewerage was developed.

[1] Rawlinson, Sir Robert, *Proc. Inst. C. E.*, **24**, 318.

At that time European sanitarians were divided into two schools, advocating respectively the "dry" and the "water-carriage" methods of collecting excrementitous matter. In the former, this matter is collected and removed in pails and in the latter it is flushed into the sewers. The former is still used in a number of European cities, but as it is not employed except on a small scale in the United States it is necessary here to give only the following brief account of it, abridged from Hering's report on European sewerage, mentioned later in this chapter. A complete summary of the subject is given in Baumeister's "Cleaning and Sewerage of Cities," where the methods of cleaning cesspools, the equipment for "dry" collection, and the disposal of the contents of the pails are treated with a detail unnecessary to repeat for American readers.

Water carriage was opposed by European chemists, physicians, and agriculturalists because of a fear of contamination of the soil by leakage from the sewers, the possible pollution of bodies of water receiving the sewage, and possible nuisances, if not actual dangers, where the sewage was distributed over land. Engineers were generally favorable to water carriage. Dr. Pettenkofer, the famous hygienist, was at first an opponent of it but subsequently became an advocate.

Dry removal accomplished its object satisfactorily, either by an immediate and thorough disinfection with subsequent removal at convenient intervals or by temporary storage with frequent removal before decomposition could be rendered injurious.

There were two common methods of disinfection. The first was the partial absorption of the sewage by dry earth, peat, charcoal, and like materials, which accelerated its decomposition and diminished offensive odors. The second was the addition of carbolic acid, chloride of lime, creosote oil, and other chemicals to the sewage.

Where there was no disinfection, the excreta were collected in a "pail," (called *fosse mobile* in France and *tonne* in Germany) made of iron or oak and provided in some cities with a tight lid having a sleeve fitting closely around the bottom of the soil pipe. These pails were collected at intervals of a day to a week and clean ones substituted for them. Where the system was conducted most satisfactorily, the pails were removed in wagons with tightly closed bodies and were carefully cleaned after being emptied. The contents were frequently used for fertilizing purposes.

The dry system, to compare favorably with the water-carriage system, must be restricted to (1) small towns, on account of the expense of cartage; (2) towns where the regular exchange of the pails can be enforced with almost military strictness, which is seldom found outside of a few European countries; (3) dwellings where water closets cannot be used; (4) localities where sewerage would be very expensive; (5) where the waste water can be led over the surface of the ground without causing offense.

There was an unusual modification of the pail system employed for some time in Paris after the cesspools became too offensive. The engineers of

the city were early advocates of water carriage for removing fecal matter, but there was great popular opposition to this although the large storm-water sewers were available for water carriage and their contents were already foul with the refuse washed from the streets. Accordingly a *fosse filtre* was temporarily used to educate the public. It was a cask of 20 to 25 gal. which retained all solids reaching it through the soil pipe but permitted the escape of the liquids into the sewer. As the liquids are the most putrescible parts of the excreta, some sanitary gain was made in this way, and, as soon as popular prejudice abated, the pail and its connections were removed and the soil pipe connected with the house drain by a few feet of pipe.

The early sewerage works in the United States are almost unknown.[1]

[1] The early American sewerage engineers of note were first engaged on such work by chance, not inclination. The list is headed by E. S. Chesbrough, who was born in 1813 and died in 1886. He became a chainman on railroad surveys when he was fifteen years old, and rose gradually in railroad engineering positions until 1846, when he became chief engineer of the Western Division of the Boston water works. He was reluctant to accept this work on account of his lack of familiarity with anything but railroad engineering, and only undertook it with the assurance that J. B. Jervis would act as consulting engineer. He remained on this work until he became city engineer of Boston, in 1850, and thus first became interested in sewerage. He resigned in 1855 to become the engineer of the Chicago Sewerage Commission and, while holding this office, he published, in 1858, a voluminous report on sewerage which was the first really important American exposition of the subject. His plans for the Chicago sewers were adopted and that city was the first important place in the country to engage in the systematic execution of a comprehensive sewerage system. This established his reputation as a specialist and he was subsequently consulted in connection with sewerage problems by Boston, Burlington, (Iowa), Chattanooga, Des Moines, Dubuque, Memphis, New Haven, Peoria, Providence, and many smaller places. He was the eighth president of the American Society of Civil Engineers.

Moses Lane, like Chesbrough, was a railroad engineer in early life. He was born in 1823 and was graduated from the University of Vermont in 1845 as a civil engineer. He was engaged in alternating periods on railroad engineering and as a teacher down to about 1857, when he became principal assistant engineer of the Brooklyn water works, under J. P. Kirkwood, and finally succeeded him. In 1869 he became a partner of Chesbrough in Chicago and thus came into touch with sewerage work for the first time. His most important plans for sewers were the systems for Milwaukee and Buffalo, but he also furnished plans for a number of smaller places. When he died in 1882, he was serving as city engineer of Milwaukee, a place he had previously held from 1875 to 1878. While his prominence as a designer of water works overshadowed his sewerage engineering, he did some of the best work of his time in the latter line.

James P. Kirkwood, born in Scotland in 1807, was one of the most painstaking engineers connected with American sewerage work. He received his technical education as an apprentice to a Scotch engineering firm, and then came to the United States. From 1832 to 1855 he was engaged mainly on railroad work, in which he rose to high office, but was also occasionally employed by the federal government. In 1855, he undertook some difficult reconstruction of water mains in New York, which attracted so much attention that in the following year he was made chief engineer of the Brooklyn water works. Before this work was completed, his health became poor, and although he was subsequently consulted by many cities and planned many important water works he was unable to accept the numerous invitations to build the works he designed. His connection with sewerage plans was usually that of a court of final jurisdiction on the designs of others, and the conservatism of his views, as expressed in the old reports by him in the library of the American Society of Civil Engineers, is in contrast with those of the contemporary American advocates of extremely small pipes and other vagaries due to Chadwick and his followers in England. His most important original work in sewerage was probably in connection with an investigation of the pollution of Massachusetts rivers, made in 1876 for the State Board of Health. He was the second president of the American Society of Civil Engineers; he died in 1877.

Often they were constructed by individuals or the inhabitants of small districts, at their own expense and with little or no public supervision. In the early part of the nineteenth century, water boards were not infrequently placed in charge of the sewerage works, which were used mainly for drainage of storm water, as cesspools were generally employed for fecal matter. The last city to banish cesspools was Baltimore; there were 80,000 of them in that city in 1879, according to a report of C. H. Latrobe, and many of them had overflow pipes discharging into the storm-water sewers, which was contrary to law. He estimated that the annual cost of cleaning these cesspools, at the contract price of $3 per load, was $96,000. As a result of the fouling of the soil by the contents of these pits, the City Health Commissioner reported in 1879 that, of 71 samples of pump and spring water taken within the city limits, "33

Of all the engineers who were prominent in planning the earliest American sewerage systems, Col. Julius W. Adams is probably the best known today, because his treatise on "Sewers and Drains for Populous Districts," published in 1880, was widely used by engineers for at least 25 years, and his professional activities in many directions, such as talking the people of Brooklyn into starting the Brooklyn bridge, made him a well-known personage. His early engineering work was done on railroads, and it was not until 1857 that he undertook the sewerage of Brooklyn, mentioned in some detail in this Introduction. The book referred to is very interesting as explaining the principles followed in the Brooklyn design, which proved to be too small in the larger sections, a fact he acknowledged without hesitation as soon as it was apparent and frequently mentioned as proof of the need of better knowledge of fundamental principles of design than he possessed in 1857. He was frequently retained later to pass on sewerage plans and wrote from time to time to the press on the subject, particularly while he was advisory editor of *Engineering News*. He was the sixth president of the American Society of Civil Engineers.

The Boston intercepting sewerage system was authorized by the legislature in 1876, on the basis of a report by E. S. Chesbrough, Moses Lane, and Chas F. Folsom, the latter the energetic secretary of the Massachusetts State Board of Health. It was designed and partly built under the direction of Joseph P. Davis, who had gained experience under Kirkwood and Chesbrough, and was a successor of the latter as city engineer of Boston. His great modesty and deep aversion to a conspicuous position in public led him to decline on several occasions a nomination as president of the American Society of Civil Engineers. The intercepting sewerage system of Boston was the first great undertaking of the kind in this country, and gave its designer an international distinction as a sewerage specialist.

The sewerage system of Providence was declared in 1881 by Rudolph Hering, after a personal investigation of such work in our cities and in Europe, to be equal to anything abroad and much better than the work elsewhere in this country. The system was designed in 1869 by J. Herbert Shedd, then chief engineer of the water works and later city engineer, and its construction was under the personal supervision of his assistants, Howard A. Carson and Otis F. Clapp, later appointed city engineer. Mr. Shedd's report of 1874 on these sewerage works was long a famous engineering document. He designed his sewers to carry off 30¼ cu. ft. per minute per acre, without entirely filling their section, and employed a runoff formula providing for the effect of different slopes, with the result that his cross-sections proved large enough for their purpose. At the request of the mayor, the system was examined in 1876 by Gen. George S. Greene, Col. J. W. Adams, and E. S. Chesbrough, who reported that it was well designed and "the details of construction . . . have been carried out with a regard to important minutiæ which is rarely seen in such work." Owing to the later prominence of the Boston work, it is only right to point out that the Providence sewers formed for some years the model American system.

Edward S. Philbrick, born in Boston in 1827 and educated at Harvard, was engaged on railroad and structural engineering mainly down to the Civil War, when he became active in the work of the Sanitary Commission and thus had his attention turned toward public

were filthy, 10 bad, 22 suspicious, and 6 good." In 1906, Hering, Gray, and Stearns reported on a general plan for the sewerage and sewage disposal of this city, which led to the construction of a comprehensive separate sewerage system and disposal works.

There was a tendency in this country, as elsewhere, to construct the early sewers of needlessly large dimensions. One of the oldest sewers

---

health matters. He was a great student of sewerage and sewage disposal problems and was occasionally engaged to report on them, but the greater part of his professional work remained in railroad and structural lines. His effect on American sewerage practice was a marked one, however, because he enjoyed writing about the subject for the press and talking about it before the students of the Massachusetts Institute of Technology and the municipal authorities of many towns and cities. Even after extensive business enterprises compelled him to give up active engineering practice, he continued to preach the gospel of good sewerage.

It would not be proper to close this brief list without a mention of the unique position held by Dr. Rudolph Hering in the history of American sewerage. Like others named, he took up sewerage work by chance. He was engaged for a number of years in supervising the construction of various municipal works in Philadelphia and in this capacity he rebuilt some of the dilapidated structures of an earlier day, constructed in many cases with porous inverts for the purpose of admitting ground water and draining cellars. This led him to investigate the reasons for the failure of these old sewers, which proved such an interesting subject that he presented the matter as a paper before the 1878 annual convention of the American Society of Civil Engineers. It will be found in the Society's *Transactions*, **7**, 252, and was not only the first, but also for many years the sole, American discussion of the design of sewer sections to carry the external loads coming on them. Although it was not so stated in the paper, the sections were designed to rest on platforms and resist the most unfavorable loadings to which such structures were exposed. The sections were, thus, somewhat heavier than would be needed under many conditions, but their publication was beneficial as counteracting a tendency at that time toward very light construction. This and other professional papers on allied subjects attracted attention to their author, and when the National Board of Health desired to make an investigation of European sewerage work, he was naturally selected, being a graduate of one of the best German polytechnic schools and familiar with American sanitary engineering practice. Bearing letters of introduction from a powerful semiofficial body, he was able to gain the close acquaintance of the English and European sewerage engineers, and to ascertain what the leaders among them thought of the many disputed features of their work. His report of his work, forming the first clear American analysis of all the main problems of sewerage and the methods of solving them, established his reputation as specialist.

Finally, the name of D. E. McComb should be mentioned as the first American engineer who dared to build large sewers of concrete. Many wished to do this, but were afraid of the quality of the concrete which would be produced as a city job, just as this feeling of distrust lasted many years longer in Great Britain and led the Local Government Board to require, in the case of reinforced concrete sewerage works, an amortization fund corresponding to a life of 15 years only. McComb was superintendent of sewers in Washington and was convinced he could get good results. In 1883, Capt. R. L. Hoxie designed a 15- by 17½-ft. concrete sewer with a complete brick lining, which was built in 1885 under McComb's supervision; this sewer was 2,500 ft. long and the maximum depth of trench was about 60 ft. Another concrete sewer designed and built at the same time had a circular section of 10-ft. diameter and a brick lining. These are the only concrete sewers in Washington with a brick lining in the invert and arch. In 1888, McComb constructed a concrete sewer 7.65 ft. in diameter and 864 ft. long, and, in connection with it, a gravel-catching basin of concrete, with an arch of 24.4 ft. and a rise of 4.5 ft., the thickness at the crown being 1.5 ft. Since that date, the use of concrete in sewer construction has been the rule in Washington, the inverts usually being lined with vitrified brick. The success of the 1885 experiment led to the use of concrete for large sewers elsewhere, and it soon was demonstrated that they were less expensive than brick sewers and could be made without serious difficulties in securing good workmanship.

in Brooklyn was in Fulton Street. Although it drained an area of less than 20 acres and was on a grade of 1 in 36 it was 4 ft. high and 5 ft. wide. For many years the largest sewer in Manhattan was that in Canal Street, built somewhere between 1805 and 1810; it was 8 by 16 ft. in section and about 1850 was in very bad condition, being referred to by engineers of that time as affording instructive information of things it was wise to avoid. Its large size doubtless was made necessary by the existence of a brook at this place which was at one time provided with plank walls and was used by small boats, as illustrated in Valentine's "Manual of New York." In some cases, the sewers were not only very large at their outlets but were continued of the same size to their heads; it was impossible to secure adequate velocity in such sewers unless they were laid on steep grades, and consequently some of them became offensive when the sludge accumulating in them underwent decomposition. In some cases, the grades were in the wrong direction; an instance of this is mentioned in a report on Boston sewerage problems made in 1876 by E. S. Chesbrough, Moses Lane, and Charles F. Folsom:

> The filling in of the old mill pond naturally necessitated the extension of the sewers of that district to discharge into the canal; and, upon closure of the canal, the sewers were intercepted by a main which now discharges on both sides of the city, very irregular in grade, and whose two outlets are materially higher than its central point at Haymarket Square, thereby causing obstructions in that whole drainage district.

Such conditions as these produced the same nuisances which were so marked in English and Continental cities in the middle of the last century. For instance, R. C. Bacot, superintendent of the Jersey City water and sewerage works, reported as late as 1865:

> The situation of these sewers and the necessity of their entire reconstruction has been brought to the notice of the proper authorities in my annual reports of the last four years, but nothing has been done by those immediately interested to remedy the evil. The outlet of the Henderson Street sewer (which is the receptacle of all these lateral sewers) being effectually closed up at the Morris Canal, no sewage matter can pass away, and consequently these sewers are almost entirely filled up with putrefying matter.

Much trouble was caused by the construction of sewers by individuals and their subsequent acceptance by the city. As long ago as 1850, Rogers, Chesbrough, and Parrott protested against such work in the following terms, in a report to the City of Boston:

> As the law now stands, any proprietor of land may lay out streets at such level as he may deem to be for his immediate interest, without municipal interference; and when they have been covered with houses and a large population is suffering the deplorable consequences of defective sewerage, the Board of Health is called upon to accept them and assume the responsibility of applying a remedy.

About the time that the last quotation was written there was considerable discussion among English engineers concerning the proper grades of sewers, and this controversy was duplicated on a less acrimonious plane in the United States. Lindley and Rawlinson were among the leading advocates of flat grades with ample provision for flushing, while Wicksteed was probably the leading champion of enough slope to keep the sewers clean without other flushing than was afforded by the ordinary maximum daily flow. The low-grade school had its way with a vengeance at Charleston, S. C., in 1857, where a sewer was built without any slope. It was 2⅝ miles long, 3½ ft. wide and 4½ ft. high, with plank bottom and brick sides and arch. Each end had a tide gate, and the tides were such that, at certain times in the day, a flushing current strong enough to move broken brick, sand, and clay could be sent through the sewer.

Some of the difficulties which the American designer of sewers, without professional treatises of much value and lacking the help of the professional societies and journals of today, encountered in the middle of the last century are set forth in a report by Strickland Kneass, Chief Engineer of the Department of Sewerage of Philadelphia, in 1857:

That portion of our charge which requires the most mature deliberation and careful examination is the arrangement of systems for drainage, with the proper proportioning of the sewers and drains constituting such systems, and has required a course of study and research that has been but little attended to in our city. It is a subject that has such a variety of elements within it as to have rendered it a matter of close investigation for a series of years in the city of London, by commissioners appointed under acts of Parliament, the results of which are very voluminous and furnish much practical information, from which may be deduced laws of great value on the question of waterflow in sewers; yet so widely do they differ from experiments on record, made upon a small scale—upon which our mathematical formulas have been established—that judgment must be exercised in their adoption, but we hope to make such experiments upon some of the most perfect of our own sewers as will enable us to draw a comparison between their practical and theoretical value. Nevertheless, we have given the subject much consideration, and believe that the principles upon which we have arrived at the proportions of those sewers and drains already designed are correct, and will be found to be fully adequate to the purposes intended, yet with a strong hope that much saving may be made hereafter by a further reduction in the proportions of sewers for a given drainage.

The foul condition of the streets of Philadelphia at that time, owing to the filth discharged or cast into the gutters, is evident from another quotation from the same report:

There should be a culvert on every street, and every house should be obliged to deliver into it, by underground channels, all ordure or refuse that

is susceptible of being diluted. The great advantage in the introduction of lateral culverts is not only that underground drainage from adjacent houses should be generally adopted, but that by the construction of frequent inlets, our gutters would cease to be reservoirs of filth and garbage, breeding disease and contagion in our very midst.

About the time Kneass was hoping that experiments would enable him to adopt smaller sewer sections, another American city was undertaking the construction of a sewerage system, based on the best English data of that period, which taught a needed lesson of the danger of constructing sewers on any other basis than a complete understanding of the requirements of the locality they were to serve. The lack of such information was pointed out by the engineer of the works in question, the Brooklyn undertaking of 1857–1859, which was designed by Col. Julius W. Adams. In his reports of that date he made these statements:

The sewers in this city already built are too few in number, and their use too restricted and with too limited a supply of water, to enable us to derive from them data of any value whatever, and the attempt to obtain it by gaging the sewers in New York City, with the imperfect system which from past necessity has prevailed there, would be attended with a great expenditure of time, and from various causes, great uncertainties would arise as to the value of the results obtained. No gagings, to our knowledge, have ever been made of sewers in this country, and very imperfect records exist of their dimensions, inclinations, and other characteristics. If gagings have been taken, they have been too limited in scale to furnish data for a system of sewers in a city of so rapid a progression in population as Brooklyn promises to be; hence, we are driven for the necessary information to those cities abroad where the subject has been forced on the public attention for a series of years

From recorded observations it appears that in a particular district, a rainfall of ½ in. in 3 hours took 12 hours before the flow in the sewer resumed its ordinary level on areas such as we are considering, and a rainfall of 1.1 in. in 1 hour and 0.3 in. in the next 2 hours occupied in discharging 15¾ hours, those points nearest the outfall draining off first, the more remote next, and some portions would be entirely clear before the water from the most remote points would reach the outfall.

The present plan is calculated for a rainfall of 1 in. in 1 hour, to be discharged in 2 hours, or a discharge of ½ cu. ft. (3¾ gal.) per second per acre of area drained.

It has been seen that we may estimate one-half of the flow of sewage, including all waste water due to 24 hours (everything but the rain) to run off in 8 hours, from 9 A. M., and that the sewage equals in amount 1¼ the water supply, or for 40,000,000 gal. water the sewage may be estimated at 50,000,000 gal., the half of which running off in 8 hours, gives 3,125,000 gal. of sewage per hour during 8 hours, which, from 12,000 acres, gives 260 gal. or 33 cu. ft. per acre per hour, or less than 0.01 in. in depth over the whole area, while the capacity of the sewer is calculated for an inch in depth.

To avoid intricacy of calculation and to err on the safe side by an excess in the dimensions of the pipes over the absolute requirements of the case, according to Adams' report, it was permissible to employ for limited areas, at the summits of branch sewers, and elsewhere as experiment might dictate, the "formula for discharge from a still reservoir," but for larger areas and mains he preferred to be governed by Roe's gagings of the London sewers. The minimum inclination given to the sewers, when running half full, is stated in Table 1, and was considered great enough to produce a velocity

> . . . which will sweep away any substance which should be found in the sewers and many which should not. This quantity of water can be introduced at any time by the process of temporary dams or gates at the manholes, producing a sudden flush or scour of the sewer by water from the hydrants.

This table is of interest in comparison with the authors' recommendations for minimum grades in Chap. III.

TABLE 1.—MINIMUM GRADES RECOMMENDED IN 1859 BY COL. J. W. ADAMS FOR SEWERS FLOWING HALF FULL

Diameter, in.	6	9	12	15	18	24
Slope, ratio	1/60	1/90	1/200	1/250	1/300	1/400
Slope, percentage	1.67	1.11	0.5	0.4	0.33	0.25

It might be added here that the recommendations for minimum slopes for brick sewers 36, 42, and 48 in. in diameter were 1 in 600, 700, and 800, respectively. By way of contrast, reference may be made to the minimum grades adopted by C. Howard Ellers, Chief Engineer of Sewers of Chicago in 1881, which were 0.2 per cent for 12- to 18-in. pipe, and 0.1 per cent for 20- to 30-in. sewers.

Although Colonel Adams was a leading student of sewerage problems and his plans were checked by James P. Kirkwood, a most careful and thorough engineer, the system proved inadequate, as is shown by the following quotation from a report of the chief engineer of the Brooklyn sewerage works on Dec. 23, 1870:

> Your engineer has been aware for several years of the importance of improving the sewerage system; and the frequent complaints of householders in certain localities of the city have caused the most careful investigations to be made from time to time. Many of the main sewers proved to be too small since the districts have been built over, and are, in not a few instances, at too low a grade. The lower portions of many districts are frequently inundated, and what is proposed is a system of interception of the sewage and storm water of the upper portion of such districts; the lower sewers will then be ample in size to deal with the volume of flow which will be due to them.

The history of sewerage works has been marked until comparatively recent times by just such results of reliance on imperfect information for design.[1] Much damage has been done by flooding cellars with storm water and sewage from surcharged sewers. Under the law of most states, which is explained in great detail in the famous New York case reported in 4 N.E. Rep. 321, if the city and the engineer follow out the legal requirements governing sewerage works, parties damaged by reason of defects due to mistakes in the design have no ground for action against the city. This shows the grave responsibility of the engineer and makes it incumbent upon him to utilize every possible source whence information pertinent to the design may be secured. The legal rule in question was stated briefly as follows by the Maine Supreme Court in *Keely* vs. *City of Portland*, 61 At. Rep. 180:

A municipal corporation is not responsible in damages for injuries caused to a person's property by the flowing back of water and sewage from a public sewer with which the property is connected, where this injury results from some fault in the location or plan of construction or in the general design of a sewer system, and not at all because of want of repair or failure of the municipality to maintain the sewer to the standard of efficiency of its original plan of construction.

A peculiar aspect of the subject was settled in 1905 by the Nebraska Supreme Court. In 1882 the city of Omaha adopted plans prepared by George E. Waring, Jr., for the sewerage of a part of the city, although the city engineer, Andrew Rosewater, protested against this action on the ground that the proposed lateral sewers were too small, being but 6 in. in diameter. The system was installed and it became necessary to build a larger sewer paralleling one of the laterals, except where it was on a steep grade. A property owner brought suit to enjoin the collection of special assessments for the larger sewer, contending that had the city followed the advice of its city engineer, it would have saved the money wasted on an inadequate system. The court ruled, however, that when

[1] Among the unique sewerage systems built in early days in the United States that in the older part of San Francisco has an exceptionally prominent place, for the methods of design and construction were marked by a complete disregard of proper engineering principles, as is evident from the following quotation from a paper by C. E. Grunsky on "The Sewer System of San Francisco" in *Trans. Am. Soc. C. E.*, 1909; **65**, 294.

"The plan . . . seems to have been to construct egg-shaped brick sewers, 5 ft. high and 3 ft. wide, in all streets and alleys where property was valuable and could afford to pay for large sewers . . . The size of sewer was frequently determined by the Superintendent of Streets, who was never a civil engineer. . . . The invert, as required by ordinance, was placed 10 ft. below street grade, generally level, or, due to the intelligence of most of the sewer contractors, a few inches lower at the downhill side of the street intersection. The sewers in the intersection might connect with other brick sewers of like size, or with larger sewers, or with small pipe sewers, according to what was prescribed at some other time, for the streets leading from the intersection."

. . . the city council, misled by the glamour of a great name, employed Colonel Waring, they did what any prudent, cautious business man would have done under like circumstances and the plaintiff cannot complain if their judgment was erroneous.

The sufficiency for its purpose of one of the largest sewers in the country was approved by the Missouri Supreme Court in the case of *Gulath* vs. *City of St. Louis*, 77 S.W. Rep. 744. This related to the Mill Creek sewer in that city, draining about 6,400 acres and begun in 1864. At its upper end it is 10 ft. in diameter and at its lower end, 5 miles distant, it is 16 by 20 ft. in section. It was designed to care for a rainfall of 1 in. per hour. Before it was built, the site of the plaintiff's store was overflowed by the creek many times, according to testimony. After the sewer was constructed, the site was overflowed but three times down to the date of the suit, and on each occasion after an unusual storm. The court ruled that such exceptional storms need not be taken into account by the engineer in designing such works.[1]

Although where a properly authorized official or committee adopts plans for a sewerage system the city cannot be held responsible in most states for damages resulting from defects of design, it has been held by some courts, as the Wisconsin Supreme Court in *Hart* vs. *City of Neillsville*, 104 N.W. Rep. 699, that the mere existence of sewers will not be considered the equivalent of a plan. In that case the court held that if a sewerage system was constructed without a properly adopted plan, the city is liable for any damages that may result from defects in it. The court also ruled that though a city was not liable for damages to private property caused by mere defects in a properly adopted and executed plan, if it was informed of such defects and the direct continuing injury to private property that would result unless they were remedied, it should exercise ordinary care to prevent such a result and was responsible for damages caused by any negligence in this respect. This ruling indicates that when a city takes over the improvements made upon a large tract of land, such as the "additions" so frequently absorbed where communities are developing rapidly, the plan and construction of the

[1] This was expressed in the following words in a preliminary report by the New York Metropolitan Sewerage Commission: "The importance of giving careful consideration to the rainfall is greater in designing collecting systems of sewerage than in providing for final disposition. The function of such sewers is not only to carry off the drainage of the houses, but to prevent accumulations of water in the streets. It sometimes happens, when excessive falls of rain occur, that sewers are surcharged. At such times the drainage of houses is interfered with and often stopped, in which case cellars may be flooded and other serious inconvenience produced. It is usually impracticable to provide combined sewers of a size and grade sufficient to carry away the water which falls in storms with sufficient promptness to insure that inconvenience from flooding shall never occur. At long intervals rainfalls of exceptional severity take place, and, to provide for these, sewers would have to be built so very large that they would represent a considerable investment over the sum required to give them sufficient capacity for all the ordinary and most of the heavy rains which are likely to fall."

sewerage systems should be very carefully scrutinized before the papers are finally signed.

In the design of sewerage systems down to a comparatively recent date, there seemed to be a strong preference for outfalls in tidal waters which were locked by flood tide, and it was by no means rare to find the outlets at an elevation which insured their submergence at mean tide. In its investigations of the sewerage systems discharging into New York Bay, the Metropolitan Sewerage Commission reached the conclusion that two opinions led to this construction, the first that the sewer bottom should be given as much slope as possible in the belief that it controlled the velocity of flow in the sewers, and the other that the wind blowing into the open ends of the sewers drove the foul air up into the streets through the perforations in the manhole covers.[1]

Another cause of flooding existed in some sewerage systems otherwise free from defects. This was the preparation of sewer plans by using the invert grade or bottom slope, for calculating capacities, instead of the hydraulic grades or slopes of the water surface in the sewers. The result of this mistaken policy in Brooklyn down to 1907, was "to produce sewers that would overflow at manholes and be, so to speak, drowned out whenever the flow approximated the maximum capacity."[2]

The United States suffered, just as England did at an earlier date, from the improper design of separate systems of sewerage in which the house sewage and rain water are kept nearly or quite distinct. Just who designed the first system of sewers for removing house sewage separately is not definitely known, but the principle was strongly advocated as early as 1842 by Edwin Chadwick. He has been called the "father of sanitation in England," and unquestionably played an important rôle in arousing that country to the need of greater cleanliness, not only in cities but also in rural districts. He was a man of convincing address, great self-reliance and enthusiasm, and strong imagination which was, unfortunately, not restrained by technical knowledge. As a result he advocated, even in meetings of engineers, so-called hydraulic principles and some features of design that were wholly incorrect, which at last

[1] That this erroneous practice had been abandoned by leading engineers before the birth of many of the readers of these pages is indicated by the following statement in a report on the sewerage of Brookline, Mass., made in 1875 by E. S. Chesbrough, W. H. Bradley, and Edward S. Philbrick: "With regard to the height of the outfall, two important reasons exist for keeping it as high as possible; *viz.*, to prevent the influx of tide water at the mouth, and to afford an advantageous connection with any intercepting sewer which may hereafter be constructed on the south side of the Charles River for Boston and vicinity. On the other hand, it is extremely desirable to keep the outlet as low as possible, both to secure an efficient inclination to the sewer and to drain as well as may be the low-lying district . . . We therefore recommend that the bottom of the outfall be placed at the level of half-tide, and that a self-acting tide gate be placed there. Should a grand scheme ever be carried out for marginal intercepting sewers for Boston, it is probable that resort must be had to pumping to make such a scheme successful, in which case the low level above named for the outlet of the Brookline sewer will not be found objectionable."

[2] *Report* Metropolitan Sewerage Commission, 1910.

resulted in his being publicly branded as a charlatan at a meeting of the Institution of Civil Engineers at which he was in attendance.[1]

The principle of the separation of house sewage from rain water, advocated by Chadwick,[2] was so meritorious for many places that it was developed along rational lines by a number of leading English engineers, notably Sir Robert Rawlinson, whose "Suggestions as to Plans for Main Sewerage, Drainage, and Water Supply," published by the Local Government Board, did much to prevent the laying of sewers of too small size and poor alignment, without proper facilities for the cleaning which is likely to be necessary in all such works.

The separate system received much study by American engineers, as was natural in view of their reliance on English practice for precedent. Fortunately, however, the difference between the character of the rainfall in England and the United States was known here and its influence on the design of sewerage works was appreciated. The English rains are more frequent but less intense, and hence our storm-water drains must be larger for like topographical conditions. Our heavier rains afford more vigorous flushing action in the sewers, so that the necessity for the rather elaborate provisions for flushing combined sewers in many European cities is not so evident here. Wherever the surface drainage could be cared for satisfactorily at a low cost without the use of large combined sewers receiving both house sewage and rain water, there was a manifest advantage in adopting the separate system, which was used at about the same time (1880) in designs prepared by Benezette Williams for Pullman, Ill., and George E. Waring, Jr., for Memphis. The Memphis system was the most conspicuous, although a comparative failure, a fact which the people of the city naturally suppressed for business reasons for many years. Colonel Waring received two patents, Nos. 236740 and 278839, issued in 1881 and 1883, for separate sewerage systems, and his use of these patents in ways which many engineers regarded as unprofessional brought severe criticism upon him.

During the summer of 1873, more than 2,000 persons died of yellow fever in Memphis. In 1878, 5,150 deaths occurred from the same cause; a rigid quarantine and sanitary regulations were enforced but the disease was merely checked and during the next year was the cause of 485 deaths. The Legislature authorized unusual taxing and adminis-

[1] See *Proc. Inst. C. E.*, 24.

[2] John Phillips, in a paper read before the Philosophical Society of Glasgow, Feb. 7, 1872 said: "The principle of drainage in towns which I advocate, and which was first proposed by me, is called the Separate System, (It is generally thought Mr. Menzies is the originator of this system, but this is not the fact.) I had matured and proposed it nearly 8 years before he resuscitated it in 1855. This was in 1847, when I was Chief Surveyor of a large portion of the Metropolitan (London) sewers . . . In my preliminary report in 1849 on the drainage of the Metropolis (London) I proposed this system for adoption. But public opinion was not then prepared for this advanced idea, and, in consequence, my proposal not only met with no support, but with considerable opposition."

trative methods in the stricken city, whose affliction aroused the sympathy of the whole nation and was largely responsible for the formation of the National Board of Health. A committee of the Board sent Colonel Waring to the city, which was inspected and surveyed under his supervision. The maximum sum that could be raised by taxation for sewers was $368,702, and sewerage was greatly needed so it was necessary to make the money go as far as possible.

Waring designed a separate system using 6-in. lateral sewers with a 112-gal. flush tank at the head of each, discharging once in 24 hours. The house drains were 4 in. in diameter. Not more than 300 houses were to be connected with a 6-in. sewer; if there were a larger number to be provided for, the pipe was to be enlarged to 8 in. toward its lower end. The main sewers were made of 10-, 12-, 15-, and 20-in. pipe; all of them were underdrained. All rain water was supposed to be excluded and the sewers were ventilated through the soil pipes in the houses. There were no manholes at first and the lampholes for inspecting the interior of the sewers were a failure from the outset, because the vertical shaft was heavy enough to crush the small pipe from which it rose. In 1880, 24.2 miles of sewers were built under Waring's direction and 2.1 miles of old sewers were bought, the 26.3 miles costing $183,086. During the next 2 years 12.3 miles were built and bought, and in that period there were 75 obstructions of the 4- and 6-in. sewers, costing $1,112 to remedy. The main lines in some places were reported by the City Engineer, Niles Meriwether, to be taxed to their full capacity. In 1883–1884, 2.3 miles were added to the system and 164 obstructions were removed at a cost of $1,982. During 1885–1886, 2.58 miles of sewers were constructed and $2,172 spent for removing obstructions. The inadequate capacity of the larger sewers had resulted in the construction of a relief sewer during this period. By that time engineers familiar with the conditions were convinced that some of Waring's favorite details had proved defective, and that the Rawlinson type of separate system, with larger pipes laid without vertical or horizontal bend between successive manholes, was preferable. The partial failure of the so-called Waring system was demonstrated, therefore, in about 5 years' experience at Memphis; this was a little longer than was required to demonstrate the same thing at Croydon, England, 30 years before the Memphis experiment. The Croydon system was made up of 6,350 ft. of 4-in. sewers, 44,436 ft. of 6-in., 6,435 ft. of 8-in., 14,100 ft. of 9-in., 2,469 ft. of 10-in., 3,324 ft. of 11-in., 12,117 ft. of 12-in., 9,518 ft. of 15-in., 1,506 ft. of 18-in., and 36 ft. of 21-in. In a period of 20 months in 1852–1853, there were 60 stoppages in the 4-in. sewers and 34 in the 6-in., but not more than one in any of the other sizes.

Waring was able, due to his pleasing personality, to use the prestige of his sanitary achievements at Memphis to impress his views regarding small-pipe sewers on a number of communities. The National Board of Health felt some distrust regarding such systems soon after its formation, and accordingly it sent Rudolph Hering to Europe on a tour of investigation, which lasted nearly a year. On his return he prepared the elaborate report on the principles of sewerage and their exemplification in the best works of Europe already referred to, which remains a thorough summing up of good practice. It is not often that an engineering monograph retains its value for more than a quarter of a century. As a result of his investigation Hering outlined the respective fields of the separate and combined systems as follows:

The advantages of the combined system over a separate one depend mainly on the following conditions: Where rain water must be carried off underground from extensive districts, and when new sewers must be built for the purpose, it will generally be cheaper. Its cost will also be favorable in densely-inhabited districts from the circumstances that the proportion of sewage to rain water will be greater, and therefore increase the sizes of the separate sewer pipes, yet without decreasing those of the rain-water sewers; while the sizes of the combined would not vary with the population, because the quantity of sewage is less than the quantity within which the amount of storm-water can be estimated. But more important is the fact that in closely built-up sections, the surface washings from light rains would carry an amount of decomposable matter into the rain-water sewers, which, when it lodges as the flow ceases, will cause a much greater storage of filth than in well-designed combined sewers which have a continuous flow and generally, also, appliances for flushing.

The separate system is suitable . . .

Where rain water does not require extensive underground removal and can be concentrated in a few channels slightly below the surface, or where it can safely be made to flow off entirely on the surface. Such conditions are found in rural districts where the population is scattered, on small or at least short drainage areas, and on steep slopes or side hills.

Where an existing system of old sewers, which cannot be made available for the proper conveyance of sewage, can yet be used for storm-water removal.

Where purification is expensive, and where the river or creek is so small that even diluted sewage from storm-water overflows would be objectionable, especially when the water is to be used for domestic purposes at no great distance below the town.

When pumping of the sewage is found too expensive to admit of the increased quantity from intercepting sewers during rains, which can occur in very low and flat districts.

Where it is necessary to build a system of sewers for house drainage with the least cost and delay, and the underground rain-water removal, if at all necessary, can be postponed.

The principle of separation, although often ostensibly preferred on sanitary grounds, does not necessarily give the system in this respect any decided advantage over the combined, except under certain definite conditions. Under all others, preference will depend on the cost of both construction and maintenance, which only a careful estimate, based on the local requirements, can determine.

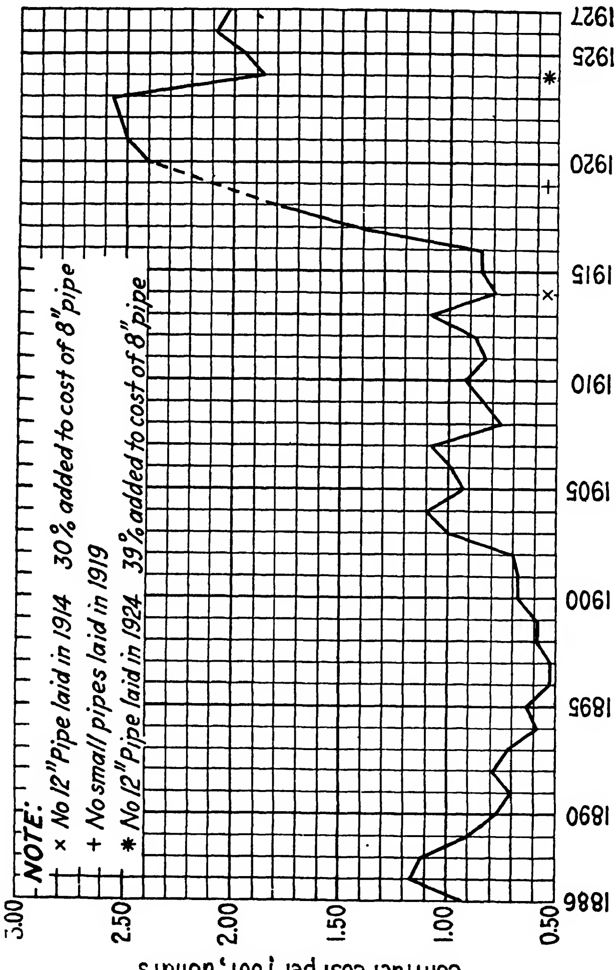


FIG. 1.—Contract costs of 12-in. pipe sewers in Newark, N.J., during a period of 42 years.

The cost of sewerage works is a subject presenting many pitfalls to those without experience, and even to those having it. The fluctuations in the rate of wages and the price of materials from year to year, the

character of the workmanship required and of the supervision by inspectors, the competence of the superintendents of construction and the introduction of labor-saving machinery, these and other factors which affect the cost of public works are not readily explained quantitatively, so that a public official or young engineer can grasp their combined effect. This effect is marked, however, as is well shown in Fig. 1, furnished by E. S. Rankin, engineer of sewers of Newark, N. J. This diagram shows the fluctuation in the contract price of 12-in. pipe sewers in 8- to 10-ft. trenches, during a period of 42 years. The costs plotted in the diagram were those of contracts for work of practically the same character and show a range from 51 cts. to $2.57. Records of this character can be duplicated in most cities where costs have been carefully kept for a long time and they prove how wary an engineer must be in preparing estimates.

The disposal of the sewage of most cities, until recent years, was carried out by the easiest method possible, without much regard to unpleasant conditions produced at the place of disposal. Irrigation with sewage was apparently practiced at ancient Athens, but there is very little definite information on any methods of disposal on land down to about 300 years ago, when sewage farming was successfully introduced at Bunzlau, Germany. The earliest municipal work of the kind in Great Britain was on the Craigentinny meadows of about 400 acres extent receiving the sewage of a part of Edinburgh for about a century. The subject of disposal received only occasional local attention, however, until the construction of sewerage systems after the cholera epidemics of 1832–1833 and 1848–1849. Owing to the small size of British streams, their pollution by the sewage discharged into them soon became objectionable. Interference with agricultural and manufacturing uses of water was apparently at first given more attention than possible danger to health. When the cholera epidemic of 1854 had been suppressed, Parliament passed the comprehensive Nuisances Removal Act of 1855, to which reference has already been made. This did not make sewage treatment compulsory, however, nor did the Rivers Pollution Prevention Act of 1876, although as early as 1865 a royal commission had reported:

First, that whenever rivers are polluted by a discharge of town sewage into them, the towns may reasonably be required to desist from causing that public nuisance.

Second, that where town populations are injured or endangered in health by a retention of cesspool matter among them, these towns may reasonably be required to provide a system of sewers for its removal.

Two methods of treating sewage came into vogue about the time of this report. The irrigation of land by sewage was the older of these

but the precipitation of the solids and some of the dissolved matter by chemical treatment and subsequent sedimentation attracted more attention owing to its exploitation by promoters as well as to the favorable opinion of it held by many careful and conservative engineers. A special committee appointed by the Local Government Board in 1875 reported on the whole subject as follows:

> That most rivers and streams are polluted by a discharge into them of crude sewage, which practice is highly objectionable.
>
> That, as far as we have been able to ascertain, none of the existing modes of treatment of town sewage by deposition and by chemicals in tanks appears to effect much change beyond the separation of the solids and the clarification of the liquid. That the treatment of the sewage in this manner, however, effects a considerable improvement, and, when carried to its greatest perfection, may in some cases be accepted.
>
> That town sewage can best and most cheaply be disposed of and purified by the process of land irrigation for agricultural purposes, where local conditions are favorable to its application, but that the chemical value of the sewage is greatly reduced to the farmer by the fact that it must be disposed of day by day throughout the entire year, and that its volume is generally greatest when it is of the least service to the land.
>
> That land irrigation is not practicable in all cases; and, therefore, other modes of dealing with sewage must be allowed.
>
> That towns, situated on the sea coast or on tidal estuaries, may be allowed to turn sewage into the sea or estuary, below the line of low water, provided no nuisance is caused; and that such mode of getting rid of sewage may be allowed and justified on the score of economy.

The density of population in England and the very small amount of land well suited for sewage farming and filtration led to particular interest in intensive methods of treatment, whereby in plants of comparatively small area the sewage was rendered suitable for a final treatment on land, which was practically compulsory for most English systems discharging into fresh water. This constraint was exercised by the Local Government Board, without whose approval money could not be raised for public works except by special act of Parliament; the Board was wedded to a final land treatment until comparatively recently. Consequently septic tanks, trickling filters, and contact beds were received with acclamation and tested on a practical scale that was unwarranted, for instance, in Germany.

The disposal of sewage in the United States did not receive so much attention 40 years ago as in England, because the extent of the offense caused by its discharge into the relatively large bodies of water was not so marked and because of the greater area of land suitable for broad irrigation or intermittent filtration on beds graded *in situ* and of relatively cheap materials suitable for the construction of artificial treatment beds in some localities where pollution was objectionable. Its importance

was foreseen by the Massachusetts State Board of Health early in the seventies, and its secretary, Dr. C. F. Folsom, made a careful study of disposal in Europe, which resulted, in 1876, in a report which was the most complete statement that had been made of the state of the art at that time. Irrigation and filtration were introduced in a few places, but it was not until certain rivers in Massachusetts became quite offensive that any work on a large scale was undertaken. The first extensive treatment plant utilized chemical precipitation and was built at Worcester, Mass., in 1889–1890, from the plans of Charles A. Allen with the advice of James Mansergh of London and Prof. Leonard P. Kinnicutt of Worcester. About the same time, the Massachusetts State Board of Health, which had been given large powers of control over the disposal of sewage, established the Lawrence Experiment Station for the study of both water and sewage treatment; the influence of the research work done there has been deep and far-reaching, being particularly noteworthy for the prominence given in early years to intermittent filtration, a method of disposal neglected in England on account of the limited tracts of land suitable for practising it.[1]

The increasing demand for sewage treatment and the impracticability of procuring sufficient areas of suitable soil for land treatment in many localities, particularly for cities even of moderate size, have led to the rather wide adoption in the last quarter-century of the more intensive methods of treatment, such as by beds of broken stone known as "trickling filters."

It was early found advantageous to prepare the sewage for treatment on land or artificial beds of sand or stone by first removing the solid matters. In England, where the sewage is much stronger than in the United States, this was and is now commonly done by chemical precipitation, but in this country sedimentation unaided by chemicals has generally been adopted, thus avoiding some expense and much trouble which otherwise would have resulted from the large volume of sludge produced by chemical treatment.

The offensive character of the sludge produced by sedimentation early led to the use of septic tanks in which the solids were rendered more or less inoffensive, but difficulties of various kinds led to the general adoption of the two-story or Imhoff tanks in which the processes of sedimentation and decomposition or digestion of solids are kept separate, the former being confined to the upper story and the latter to the lower compartment. This type of tank was strongly advocated by Hering, who was instrumental in having it adopted in this country, the earliest installation being that at Madison-Chatham, N. J., in 1911.

[1] Although not used widely, this method of disposal had been employed for a number of years and was described in detail in Bailey Denton's "Ten Years of Intermittent Downward Filtration," which was published in 1881.

At Baltimore, since 1914, these two steps in the preparatory treatment have been carried out in independent tanks, the process being called "separate sludge digestion" to distinguish it from the two-story tank process. Recently, separate sludge digestion has become more popular among engineers and effort is now being made to develop it so as to control and encourage the natural agencies in an effort to secure more rapid and complete digestion and to avoid odors which have resulted in some cases in the past.

The most recent process to be developed consists of aerating the sewage in the presence of previously aerated sludge. The suspended solids are largely oxidized or collected into a rapidly settling floc by this treatment, which is known as the activated sludge process, and a high degree of purification is secured. The process was first applied in a municipal plant for treating sewage at San Marcos, Tex., in 1916. T. Chalkley Hatton, for many years Chief Engineer of the Milwaukee (and later the Metropolitan) Sewerage Commission, carried out an extensive research program at Milwaukee, perfecting the process as a method of sewage treatment and endeavoring to produce a sludge which would possess marketable fertilizing value. In the Milwaukee plant, as put into operation in 1925, special attention was given to the dewatering or drying of sludge and this sludge has been successfully converted into a commercial fertilizer. Many other notable installations of activated sludge plants have followed, including the plant at Indianapolis and the North Side plant now under construction (1927) by the Sanitary District of Chicago.

With the increased understanding of the chemistry of colloids and of the effect and control of ionization phenomena, the problem of sewage disposal is being attacked in a more scientific manner than ever before in its history, and experiment is playing a rôle of increasing importance in coordinating theory and practice. The advance in the art of sewage treatment during the last quarter-century is well illustrated by a comparison of the space required to treat the sewage from a city of 600,000 population by three different installations representative of this period and roughly estimated by the authors.[1]

Settling tanks, sludge beds, and intermittent sand filters	800 acres, or 140,000,000 cu. ft.
Imhoff tanks, sludge beds, and trickling filters	60 acres, or 17,000,000 cu. ft.
Activated sludge plant complete	10 acres, or 5,000,000 cu. ft.

Disposal by dilution has been retained in greater favor in the United States than in England because of the larger bodies of water available for receiving the sewage. The first comprehensive American

[1] *Eng. News-Record*, **92**, 1924; 695.

study of the subject was made by Hering for Chicago in 1887. He recommended the diversion of the sewage from Lake Michigan and its discharge into the Desplaines River which flows into the Illinois, a tributary of the Mississippi. For rendering the sewage unobjectionable, he advised that a large volume of water be drawn from the lake to provide adequate dilution.

This is probably the most conspicuous illustration of the dilution method of sewage disposal because of the great population served, the artificial works required, the radical procedure of actually diverting the sewage and the storm discharge from its natural outlet in Lake Michigan, the source of Chicago's water supply, and the continuous controversy which has been waged over the propriety of this procedure. That it has been remarkably successful in protecting the water supply and bathing beaches from gross contamination cannot be denied and it is hard to picture the conditions which would have resulted had any other known plan been adopted.

The growth of the Sanitary District of Chicago has been great and rapid and the quantity of polluting matter discharged now exceeds the capacity of the dilution works, including the Desplaines and Illinois Rivers, to satisfactorily disperse, assimilate, and purify it. To remedy this situation, the Sanitary District has embarked on a program of treatment-plant construction surpassing in magnitude any other thus far attempted.

Many studies have demonstrated that so far as the prevention of nuisance is concerned, disposal by dilution is the most economical method of disposal of sewage for many cities. In 1924, 88 per cent of the population in cities of 100,000 or over in the United States disposed of their sewage by dilution without prior treatment.

Caution should be exercised, however, in adopting sewage disposal by dilution, because there may be grave danger in discharging raw or merely screened and settled sewage into rivers or lakes furnishing water for potable purposes. It may be and probably is less expensive to obtain a potable water by filtering a sewage-contaminated supply than to treat the sewage so elaborately that there is no danger attending the discharge of the effluent into streams from which water supplies are drawn, but the extent to which water may be contaminated without placing an undue burden upon water filters and disinfection plants is undetermined.

It is generally conceded that it is not safe to rely exclusively upon disinfection of sewage-contaminated waters, and that the filtration of such waters with subsequent disinfection is a much more dependable means of protecting the public health. Even this may not always be an adequate safeguard. The extent to which a river or lake used as a source of domestic water supply may be contaminated with sewage

without jeopardizing the safety or palatability of the treated water, is an exceedingly important subject. There are doubtless those who, for sentimental reasons, would go so far as to require that all sewage be excluded from such waters and others would admit only such sewage effluents as had received elaborate treatment. While theoretically desirable, such extreme measures would entail enormous financial expenditures which may not in all cases be justified by the dangers encountered in the use of a properly filtered and disinfected water, even though somewhat contaminated before treatment. Others would go so far as to throw almost the entire burden upon the water-treatment plant, with slight regard to the possibility of its occasional temporary failure to afford complete protection. The sanitary engineer who neglects to work for the best interests of the public health falls short of the full discharge of his professional obligations, but it is wise to keep in mind a fact stated as follows by *Engineering News:*

> We know of many instances in which business men distrust engineers and pin their faith to so-called "practical" men, largely because of unfortunate experience with engineers who appeared to think that the question of cost was no part of their concern.

The legal dangers of attempting to discharge sewage into a small body of water must be considered in the design of sewerage systems. In *Sammons* vs. *City of Gloversville*, the New York Court of Appeals decided that although the city exercised a legitimate governmental power for public benefit when it built its sewers, it had no charter rights to discharge sewage into a brook in such a way as to injure the plaintiff's lands below the point of discharge. Even where a city had statutory rights to construct sewers emptying into a creek, whereby a nuisance was created, the Alabama Supreme Court held in *Mayor, etc., of Birmingham* vs. *Land*, 34 S. Rep. 613, that the owner of a riparian farm below the sewer outlet was entitled to damages. The Maryland Court of Appeals similarly decided the case of *West Arlington Imp. Co.* vs. *Mount Hope Retreat*, 54 Atl. Rep. 982. The fact that a water course is already contaminated does not entitle other persons to aid in its contamination or prevent those thereby injured from recovering from them damages for the injury; Ind. Sup. Ct., *West Muncie Strawboard Co.* vs. *Slack*, 72 N.E. Rep. 879.

The case of Waterbury, Conn., was of much interest for many years because of the protracted fight made by the city against building purification works in accordance with a decree of the Connecticut Supreme Court going into effect on Dec. 1, 1902. In one of the subsequent decisions in this litigation, the court stated that the construction of the Waterbury sewers in 1884, in accordance with the terms of its charter, was lawful and that their construction to discharge sewage into the

Naugatuck River gave nobody cause of action. The sewers could be used for that purpose without any invasion of the rights of owners of riparian property below the point of discharge. But when the city discharged sewage into the river in such quantities and in such manner that it was carried without much change to the property of a manufacturing company, thereby producing a public nuisance to the company's special damage, the city was held to make a public nuisance of its sewerage system. Each day such an unlawful act was repeated the company suffered a fresh invasion of its legal rights, according to the court.

While these introductory notes are intended merely to show how the principles of sewerage and sewage disposal became established on a firm footing in engineering practice and not to review the development of the details of the subject, particularly the more recent development, it should be stated here that progress has been wonderfully rapid. When the reason for this is sought, it will be found in that admirable spirit of good will and co-operation existing among American engineers, which not only finds expression in the work of the engineering societies but also in the close and friendly contact maintained by engineers in this country with one another and with the engineers of other countries. This has been a good influence on American sanitary engineering, for it has led to friendly personal relations, open minds, and a recognition of the work of others by giving credit where credit is due, which have combined to concentrate attention on those subjects where progress was most needed and to prevent the needless duplication of effort in striving for the same goal. So long as this spirit persists, American sewerage engineering will go forward buoyantly.

CHAPTER I

THE GENERAL ARRANGEMENT OF SEWERAGE SYSTEMS

It has been pointed out in the introductory chapter that many of the troubles with early sewerage systems were due to an underestimate of the amount of the rainfall reaching the sewers and an overestimate of their capacity. At a later period, another error of judgment was made often, which is causing trouble now; this was the failure to plan works capable of extension on the original lines after the cities had grown much larger. There is a limit, of course, beyond which an engineer is not justified in making allowances for the requirements of the future, but the former neglect to look ahead for more than a relatively few years has recently made very expensive works necessary in a number of cities. It is not wise to place a heavy financial burden on the present generation for the benefit of those to come, but if future expenses can be reduced by careful planning today, without appreciable additional cost, such a course is manifestly the right one to adopt.

One cause of the confusion that sometimes arises in considering sewerage plans, is a failure to recognize that there are distinct general arrangements of sewers and there are several distinct classes of sewers, each having a main purpose.

CONDITIONS GOVERNING A SEWER PLAN

The general outline of a sewerage system is governed by two prime factors, the topography of the city and the place of disposal of the sewage. The two are sometimes so simple in their effect that the general plan to be followed is self-evident, but in other cases they have complex interrelations that require protracted study before the best plan can definitely be determined.[1]

Influence of Disposal Methods.—There are three general methods of disposal that affect the design of the sewers.

The first method is to discharge directly into a river or other body of water on the shore of which the city lies; probably the Borough of Manhattan offers the best example of this, with its main sewers running east and west to numerous outlets on the North and East rivers.

[1] The first general discussion of this subject in English was apparently in Hering's 1881 report to the National Board of Health.

The second method of disposal is to intercept the sewage and a part of the storm water and carry it to a point in the adjoining body of water where it will not cause trouble; this may not be necessary at first, but in most cases it is inevitable if the city grows as rapidly as do most American municipalities, and attention must be paid to it, particularly to the future desirability of providing for the local discharge of a part

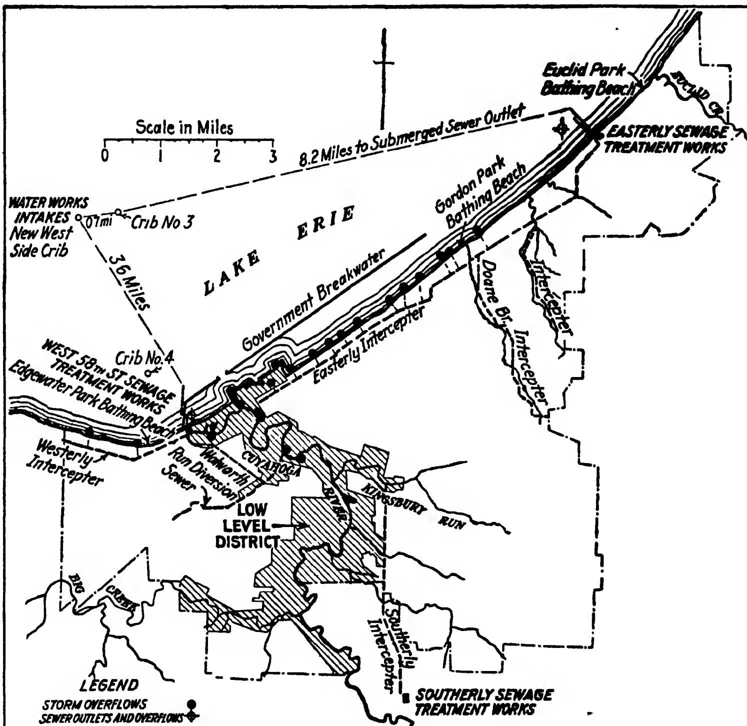


FIG. 2.—Cleveland intercepting sewer system.

of the storm flow. The Cleveland system, shown in Fig. 2, is an example of this intercepting plan.

The third method of disposal is to give some treatment to the sewage which will materially change its character and to discharge it into a body of water. This necessitates delivery of the sewage to treatment works, suitable sites for which are difficult to procure in many cases, particularly where the country is well built up, as enough open land properly located is not available in the city, and neighboring towns object to the plant being located within their limits. The separation of

storm water from the house sewage often becomes financially advisable, so as to permit the former to be discharged by short, direct lines into a river, lake, or bay nearby, and also to keep down the cost of the long sewer to the disposal works, and in some cases the disposal costs as well. In the case of combined sewers, the same end is attained by making provision at one or more points for the discharge of the storm flow in excess of a predetermined volume, through overflow weirs or chambers into channels or other outlets leading directly to the river or lake. The early flow of storm water carries a large amount of organic matter from the streets into the sewers and takes into suspension some of the matter deposited previously in the sewers, and its treatment is often considered as desirable as that of the house sewage.[1]

The design of the overflow chambers is thus an important matter. It may be found practicable to permit a large proportion of the sewage to escape through some of them in the early years of their use, but later, owing to a change in the character of the body of water receiving this excess storm flow, or the greater impurity it may then possess, its delivery to the disposal works may prove desirable. While it is inadvisable in many cases to give the outfall sewer to the works a very large capacity to provide for such future possibilities, owing to the heavy fixed charges such construction will cause, it is often desirable to consider future requirements with particular care in planning the overflow chambers, in order that their reconstruction or modification may not cause difficulties in the operation of the system out of all proportion to the cost of the work.

Influence of Topography.—The topographical features of a city also have a marked influence on the design of a sewerage system. In a large city situated on a flat plain without any neighboring river or lake into

[1] The Local Government Board of England generally required until recently that any increase in the flow in combined sewers up to three times the normal dry-weather rate should be treated like house sewage, and that six additional dilutions should be passed through "storm filters" of gravel, broken stone, or clinker. In the fifth (1908) report of the Royal Commission on Sewage Disposal, these requirements are criticized thus: "These requirements should, we think, be modified; they are, in our opinion, not sufficiently elastic, and experience has shown that special storm filters, which are kept as standby filters, are not efficient. We find that the injury done to rivers by the discharge into them of large volumes of storm sewage chiefly arises from the excessive amount of suspended solids which such sewage contains, and that these solids can be very rapidly removed by settlement. We therefore recommend, as a general rule, that: (1) Special standby tanks, two or more, should be provided at the works and kept empty for the purpose of receiving the excess of storm water which cannot properly be passed through the ordinary tanks. As regards the amount which may be properly passed through the ordinary tanks, experience shows that in storm times the rate of flow through these tanks may usually be increased up to about three times the normal dry-weather rate, without serious disadvantage. (2) Any overflow at the works should only be made from these special tanks, and this overflow should be arranged so that it will not come into operation until the tanks are full. (3) No special storm filters should be provided, but the ordinary filters should be enlarged to the extent necessary to provide for the filtration of the whole of the sewage which, according to the circumstances of the particular place, requires treatment by filter."

which the sewage may be discharged without elaborate treatment the radial system may prove best. This has its most elaborate development in Berlin, where it was introduced by Hobrecht. The city is divided into a number of sectors and the sewage of each sector is carried outward by pumping to its independent disposal farm, or the trunk sewers of two or more sectors may be combined and carried to a farm. There were eight farms in 1910. The advantage of this system is that most of the sewers are likely to be of adequate capacity for a long period, and the large, expensive sewers are reduced to their minimum length.

The sewage of each district is pumped through force mains to the irrigation farms, which, with such an arrangement, can be divided around the suburbs of the city. The water courses and, in part, the low ridges, form the limits of the districts, whose number has now risen to 12, and whose size varies between 672 and 2,128 acres. The pumping station is located at the lowest level possible, and in only one district is an intermediate pumping station necessary. The advantages resulting from this arrangement are so great that the increased cost of pumping due to the division of the pumping capacity is unimportant and can be counterbalanced by the greater security of operation. The overflow works, for which the water courses of the city act as outlet channels, form an important feature of the system.[1]

In most cases, such an arrangement is rendered impracticable by the existence of hills, water courses, and other topographical conditions. Usually, moreover, old sewers complicate the problem, for it is always desirable to utilize existing structures so far as practicable. Only in rare cases does the engineer have an opportunity to design a complete sewerage system for a large city, as was the case in New Orleans and Baltimore.

In Baltimore, where the sewage had to be taken 5¾ miles outside the city for treatment, there was opportunity for collecting the storm water separately, for there was no objection to its discharge into the nearest water courses adapted to receiving it. The city is intersected by four streams, which discharge into branches of the Patapsco River. One of these streams receives so much foul runoff that it has been covered over; the others are open. The Patapsco and its branches are tidal arms of Chesapeake Bay. The drainage area was divided into 28 districts, and the storm-water drains in each one were planned independently of the rest, to fit the topography and arrangement of streets in the best way. These drains were kept as close to the surface as possible, in order not to force the sewers so low that it would be difficult to connect the houses with them. In one low-lying district where the drainage problem was particularly difficult, the plans called for raising the street grade and building a drain to carry the storm water into the

[1] Frühling, "Die Entwässerung der Städte," 1910.

Patapsco River instead of a nearer stream which was liable to have its surface raised considerably during floods, a condition which might cause a surcharge of the drains emptying into it.

The removal of the house sewage was a much more complicated problem. Part of it comes from districts which are high enough to enable the sewage to flow by gravity to the treatment works, but a large part has to be pumped. The contour separating these two service districts was determined by two factors, the elevation at which the sewage must

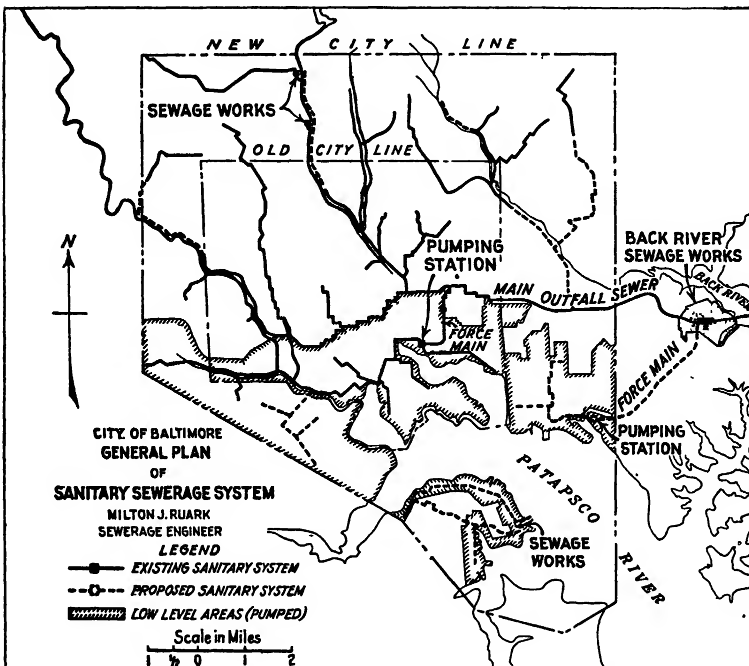


FIG. 3.—Baltimore main sewer system.

be discharged at the treatment works and the minimum safe grade of the outfall sewer from the city to the works. The accompanying plan of the main sewers (Fig. 3) shows where the outfall sewer reaches the eastern boundary of the city and is continued through it toward the western boundary as a high-level main, receiving all the sewage that can be delivered by gravity to the disposal works. The sewage of the low-lying portions of the city is collected by four main sewers, on two of which are small pumping plants to lift the sewage enough to prevent excessive depth of trench, which is undesirable on account of the high

cost of construction in deep trenches in water-bearing soil, and the difficulty of connecting the tributary sewers satisfactorily with deep-lying mains. All these mains run to a station containing five pumps, each with a nominal rating of 27,500,000 gal. a day against a total head of 72 ft. These pumps force the sewage through two lines of 42-in. cast-iron mains 4,550 ft. long into a sewer about 1 mile long, discharging by gravity into the main outfall sewer.

Between the arrangement of sewers in the Borough of Manhattan, discharging both storm water and house sewage through short lines and many outlets into the nearby rivers, and the arrangement at Baltimore, with its separation of the storm water and the sewage, its high and low levels, pumping stations, long outfall sewer and elaborate sewage treatment works, there is an infinite variety of combinations practicable. In every case, however, the topography suggests the natural drainage and the street plan exercises a more or less strong modifying influence. One of the most experienced old-school American engineers, William E. Worthen, the seventeenth president of the American Society of Civil Engineers, when he was retained to plan important sewerage improvements in Brooklyn, had constructed a large relief map of the district in order that he might see the whole topography of the area clearly while considering the existing troubles and the various remedies for them. While such a map is unnecessary in most cases, of course, topography is sometimes far more important than street plans. In every case special attention should be paid to the low-lying districts, for it is there that the largest sewers must be built in many cases, and the difficulties of construction are the greatest. It may be found advisable to reduce such work to a minimum by constructing an intercepting sewer at a somewhat higher level and thus restrict the construction in the low-lying sections to small sewers only deep enough to serve the property of that district.

Another influence of topography on sewerage plans, often overlooked, was stated as follows by Dr. Hering in his report of 1881 to the National Board of Health:

> In case of sudden showers on a greatly inclined surface which changes to a level below, the sewers on the latter will become unduly charged, because a greater percentage flows off from a steeper slope in a certain time. To avoid this uneven reception, the alignment should, as much as possible, be so arranged as to prevent heavy grades on the sloping surface, at the expense of light ones on the levels. In other words, the velocity should be equalized as much as possible in the two districts. This will retain the water on the slopes and increase its discharge from the flat grounds, thus corresponding more to the conditions implied by the ordinary way of calculating the capacity of sewers. It will therefore become necessary not to select the shortest line to the low ground, but, like a railroad descending a hill, a longer

distance to be governed by the gradient. This does not necessarily imply a longer length of sewers for the town, because more than one sewer for a street is not required by it.

Still another decided influence of topography is shown where the configuration and surroundings of the city are such that it is advisable to employ combined sewers in all parts of the city down to the lowest contour line which will permit storm-water overflows to be used. This is the rule adopted by E. J. Fort for the new sewerage works of Brooklyn. Below this contour line, the storm-water sewers are run at a higher level than the house sewers, so as to have a free outlet to tide water, and the house sewage of the low districts is pumped to points of disposal.

In some cities, the revision of old sewerage systems has been coupled with the protection of low-lying districts against flooding, as in Washington. In the original plan for the improvements, two levees with a total length of 4,000 ft. were proposed for the protection of about 900 acres of water-front property, but later a large amount of filling of park and city property and raising of street grades was substituted for the original project. The city, which uses the combined sewerage system, now has intercepting sewers around it, and a few through it in order to take advantage of topographical conditions which enable the sewage of the higher areas of the city to be kept out of the low-lying areas. All the dry-weather sewage is delivered to a pumping station which discharges it through an outfall sewer 18,000 ft. long into the Potomac River about 800 ft. from shore. A considerable quantity of storm water from low-lying areas is also pumped at this station, but only into the Anacostia River on the bank of which the plant is located.

After the most favorable location of the main lines of sewers has been determined, the desirability of minor changes of position in order to avoid needless interference with travel through busy streets should receive attention. The construction of a sewer in a narrow or crowded street costs the community a considerable sum in indirect damages and directly affects those having places of business on the street.

CLASSIFICATION OF SEWERS

Until quite recently there was considerable confusion in the terms used to designate different classes of sewers. A classification is necessary because it affords the only convenient means of discussing collectively the features of sewers for the same purpose in different parts of a system or in different cities, but the different classes necessarily run into each other somewhat so that no clear line of distinction between some of them is practicable.

Building connections or house sewers are the small-pipe sewers leading from buildings to the public sewers. Strictly speaking, the

house sewer is the nearly horizontal piping in a cellar into which the soil and waste pipes discharge, but custom has extended the use of the term to the connection between the building and the sewer. In some cities, they are put in and the connections with the public sewers are made by plumbers, but in other places the part of the work under the street as far as the property line, or even the whole connection from the sewer to the building is laid by the city. City construction is advocated by many engineers on the ground that it is necessary in order to prevent injury to the sewers where the connections with them are made and to insure good workmanship on the connection in order to avoid digging up the streets to remove obstructions caused by poor construction. On the other hand, where the municipal regulations governing building connections are properly drawn and rigidly enforced by competent inspectors, good workmanship can be secured from private contractors.

So much trouble has been caused by the breakage of and leakage from ordinary sewer pipes in or near the places where they pass through walls that a rule has been issued in most large cities requiring cast-iron pipe to be employed for the sewer for a distance of several feet outside the walls. Even if cast-iron pipes are used, care must be taken to have them properly supported so that they will not be cracked by settling. Where there is danger of a settlement of the foundations of the building, local conditions must determine the best construction.

The minimum size of house connections is 4 in., for smaller sizes are likely to become clogged frequently, but 5- or 6-in. sizes are considered better practice by many engineers, the latter being commonly adopted in the larger cities. The minimum fall for a connection is usually fixed by a city regulation, and less than ¼ in. per foot is rarely permitted. Where the building connection must carry rain water as well as house wastes, city regulations sometimes fix the size of the pipe by the size of the lot and an assumed rate of rainfall. In New York, for instance, the basis of calculation is a rainfall of 6 in. per hour with the pipe running nearly full at a minimum velocity of 4 ft. per second. These figures lead to large drains in the case of buildings covering considerable area, and in such cases two or more drains are often run to the street sewer. The capacities of pipes are discussed in Chap. II.

Owing to the annoyance which may be caused by a stoppage of a building connection, just as much care should be paid to its location and construction as is given to a street sewer. It should run on a uniform grade and straight alignment, if possible, and where a bend must be made it is generally considered desirable to use curved pipe if the deflection is more than 6 in. in 2 ft. Some engineers recommend inspection holes at every angle in a building connection; these are shafts of small sewer pipe rising from a tee in the connection, and they are objectionable because their weight often breaks the pipe below and their tops are easily

damaged by lawn mowers and children. In any case there should be a clean-out hole on the sewer just inside the house, where a cleaning rod or heavy wire may be pushed into the pipe to determine the location of, and if possible push along any stoppage.

The building connection enters the sewer at a branch, if the sewer is of pipe, or a slant if it is of masonry. Where the sewer is in a deep trench, a vertical pipe called a chimney, encased in about 6 in. of concrete, is sometimes run up from every branch or slant by the contractor. It ends at a uniform depth below the surface, such as 13 ft. in the Borough of the Bronx, and the house sewer is connected to its top. In any case, the angle of the entrance should not be more than 45 deg., for the splashing of the hot liquid house wastes containing grease on the cool walls of the sewer is liable to cause a heavy, tough coating on the latter, which reduces the discharging capacity, and this splashing will be less if the sewage enters at an easy angle rather than at 90 deg. For the same reason, it is well to give only a moderate vertical angle to the inlet into the sewer, and to place the slants in brick sewers in such a position that they do not allow the house sewage to trickle over much of the wall before mingling with the dry-weather flow. A one-eighth bend may be used next the branch or slant in order to give the line a rectangular position with respect to the sewer.

In every case, care should be taken that the building connection is so constructed that there is no danger of sewage backing through it into the cellar.

Lateral Sewers.—A lateral sewer is a sewer which does not receive sewage from any other common sewer. Experience has shown, as explained in the Introduction, that, preferably, they should not be less than 8 in. in diameter for a separate system, for a smaller sewer is liable to become clogged, although, in small communities, 6-in. pipes are sometimes used with success. There is a marked tendency to consider 12 in. as the smallest diameter for a combined sewer or storm-water drain. Theoretically, anything liable to cause clogging should lodge in the building connections, but theory is not so good a guide as practice in this case. The upper ends of the laterals are termed dead ends.

Manholes affording access to sewers are described in Chap. XV. No sewer which is so small that a man cannot enter it should have any curve or change in grade between manholes, as, otherwise, cleaning it may be difficult. Large sewers may be given such curves and changes in grade as conditions demand, but with small sewers the changes should be made by channels in the bottoms of the manholes, the loss of head due to the turning being compensated sometimes by an increased fall in the manhole.

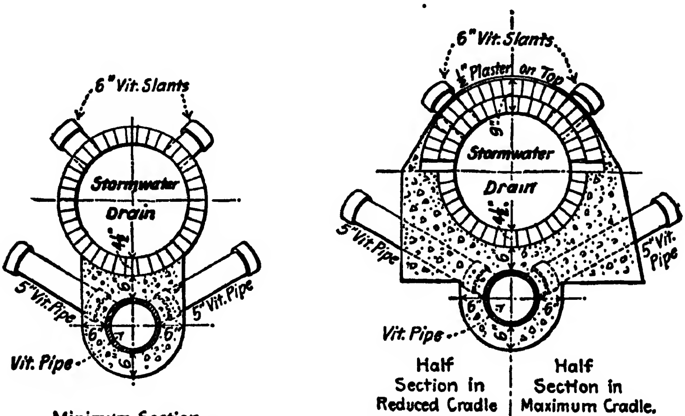
The depth of the laterals below the street surface should generally be as little as possible and still give adequate sewerage to the houses. This depth varies greatly, for in a city like New Orleans few houses have cellars and therefore shallow depths are sufficient, whereas in Boston and New York deep cellars prevail and consequently the sewers must be much lower. Where a sewer is laid in a street running along a steep hillside, it sometimes has to be given unusual depth to receive the sewage from the houses on the lower side. One long sewer in a flat street is often more expensive than two short ones having the same total length but discharging in opposite directions. The long sewer is likely to have a flatter grade and to require a deeper trench at its lower end. Where the territory is flat and the ground-water table close to the surface, it may be necessary in order to give the laterals sufficient fall, to construct them below the water table; in such a case and where, for other reasons, a sewer is abnormally deep, risers are sometimes run up so that the building connections can be laid to them above the ground-water level and without necessitating deep excavation.

The proper capacities and the minimum grades of sewers are discussed in detail in subsequent chapters. It is only necessary to state here that the true grade of a combined sewer or storm-water drain is its hydraulic gradient. There has been an unfortunate tendency to use for computations the grade of the invert or the crown of the arch as the grade of both combined and separate sewers, a practice which has led to serious trouble in some cases.

There is always uncertainty regarding the volume of ground water which may leak into a sewer, and a large group of small buildings or a schoolhouse may concentrate at one locality a quantity of sewage which cannot be foreseen. The difference in cost between small sewers of different diameters is not great, and in order to obtain satisfactory velocities with the volumes of sewage which may reasonably be expected, and still have capacity for an occasional unexpected condition, lateral pipe sewers are sometimes figured as running half full when carrying the maximum quantity of sewage which it is assumed will reach them. Some engineers have continued this policy until pipes as large as 18 in. have been reached and in computing this size they have provided for a depth of only seven-tenths of the diameter, using larger sizes when the quantity to be carried exceeds the capacity at this depth and in each case using seven-tenths of the diameter as the position of the hydraulic gradient. It appears to the authors to be more logical to make allowances for such unusual increments in flow when determining the maximum quantity to be provided for, bearing in mind that the size of pipe must be such as to provide self-cleaning velocities under usual conditions of flow, and to figure the sewers as running full.

According to the accepted formulas for computing velocity of flow, the velocity in a circular sewer half full is the same as when the sewer is full. Accordingly, no reduction of velocity is expected if sewers are designed to flow half full. But as the design is based upon the quantity of sewage to be expected in the future, there will be a period of considerable length when the ordinary flow will not approach mid-depth, and times when the depth will be very slight. The velocities of flow will then be very much less than those upon which the design is based.

Where the separate system is used and a large storm-water drain runs through a street, this may be in the way of the building connections



Side manholes affording access to the separate sewer from the side instead of the top are used in this form of construction.

FIG. 4.—Standard arrangement of separate sewers and storm water drains, Philadelphia.

from one side and may make it difficult to connect the houses with a single lateral. It is sometimes advisable in such cases to run a lateral on each side of the street, as in Washington. This results in an additional conduit in the street, but it eliminates a large amount of troublesome work with small pipes crossing the street, which may interfere with the laying of future conduits. If the street is wide and the lots have a small frontage, the double laterals may even be cheaper.

While the position of the laterals in the street is influenced by local conditions, they are usually placed in the center, thus equalizing the length and cost of building connections which are built wholly or in part by the abutting property owners. This location favors a minimum depth of sewer to provide proper fall for the building connections. As sewers are usually laid much deeper than water and gas mains, they should be kept at least 6 ft. from the latter, if possible, so as to avoid

the danger of injuring such mains during construction. Where the line is on one side of the street and property owners pay for the actual length of their house connections, those on one side have a financial advantage over the others. Where the drains are laid by the city it may be possible to arbitrarily divide the cost of the two opposite connections and charge one-half to each abutter.

In Philadelphia, standard general sections for installations on the separate plan have been adopted by George S. Webster, Chief Engineer of the Bureau of Surveys. The relative position of all conduits under 3 ft. diameter is shown in Fig. 4; the general arrangement for larger conduits is much the same. Slants and pipes for building connections are put in every 15 ft. The minimum thickness of concrete between the drain and the sewer is 6 in., except in rock excavation. With these sections, the filling over the top of the drain is at least 3 ft. deep. Many engineers prefer to have the sewers at one side of the drains, in order that they may be reached readily; this requires a wider trench than where the two-story arrangement of Fig. 4 is employed.

Sub-main Sewers.—A sub-main or branch sewer is a sewer into which the sewage from two or more lateral sewers is discharged. Experience has indicated that these long branches, which often lie on the boundary between pipe and masonry construction, are quite troublesome to arrange, and that defects in their plans are as likely to arise as in the design of the other classes. The reasons for this are several. In order to economize in the cost of construction of both sewers and building connections, the depths of the sewers below the surface should be as small as possible, but in order to carry off the sewage from the laterals, the sub-mains must necessarily be deeper than local house drainage alone demands. The grade must be steep enough to give an adequate velocity and flat enough to keep the points where the branches enter the trunk sewers sufficiently high to allow the latter, in the case of combined sewers, to discharge their dry-weather flow into intercepting sewers. Furthermore, sub-mains serve relatively small districts, and if storm water is carried by them, a material increase in the extent of impervious territory may make such a change in the maximum amount of runoff reaching them in short periods of time that they will become surcharged before the capacity of the large trunks is reached. On the other hand, sub-mains of large capacity but carrying small quantities of sewage are likely to collect sludge on the inverts, owing to the low velocities. Consequently, the engineer has to select a size and grade which reduces the total of disadvantages to a minimum. In such cases, the egg-shaped sewer is sometimes employed to advantage, owing to the small channel at the bottom of the section, which usually has a radius of about one-fourth the maximum width, and a total height about one and a half times the maximum width.

In many cases, it is impracticable to connect the laterals to the lower portion of a sub-main without using very deep trenches for the lower parts of the laterals and their house connections, or else keeping the laterals at a higher elevation and allowing them to discharge into the sub-main through a drop manhole, a special structure described in Chap. XV. The choice between the deep lateral and the drop manhole depends primarily on their relative cost, and in determining costs the expense of deep building connections as well as laterals should be considered.

It was pointed out by Hering, in 1881, that an axiom of sewerage design was that a sewer of X times the capacity of another does not cost X times as much money, and it is therefore desirable to lead as many laterals together into sub-mains as possible. This also gives the laterals better grades, as a rule.

Another thing to be considered with low-lying sewers in districts where high buildings are carried on wood piles was brought out as follows in a report on the sewerage of Hoboken, made in 1912 by James H. Fuertes:

> Many of the large and fine buildings in Hoboken rest upon wooden piles, and these will remain safe and stable so long as the piles are kept submerged below the ground-water level. If the ground-water level were to be lowered below the present prevailing height, then trouble would be sure to be felt in a comparatively short time, by the rotting of the piles and grillages, the crushing of the timber, and the settlement of the buildings. If all the sewers and their connections were perfectly tight and would remain so, there would be little likelihood of danger from this cause in securing good deep-cellar drainage. I am quite certain, however, that sewers cannot be maintained in such a condition in Hoboken.

This recommendation is confirmed by observations in New York and portions of Boston, where the construction of subways and sewers has lowered the ground-water level in places and comparatively new foundation piling has rotted away.

Main Sewers.—The main or trunk sewers are the main stems of the sewerage network; in small cities there may be only one, but in large cities there may be several, sometimes uniting where the general arrangement of the system is that of a fan and sometimes discharging independently into rivers, lakes, or ponds, like the trunk combined sewers of New York and most storm-water drains everywhere.

There is, of course, a great difference in the design of the main sewers of separate and combined systems. Where storm water enters into consideration it usually exceeds the amount of house sewage so greatly that the required capacity of the sewers is determined by it. The only influence of the house sewage on the design is to govern to some extent the shape of the invert, in order that the channel for the dry-weather flow may be such that the velocity during rainless periods will be main-

tained within desirable limits. The flow in sewers is discussed in the next chapter.

The size of main sewers receiving house sewage only may be selected on somewhat narrower lines than the size of the laterals and sub-mains, because it is hardly probable that all these small sewers will receive more sewage than the expected future maximum. Nevertheless, in many cases, the maximum assumed quantities are not more than about seven-tenths of the greatest capacity of the sections which have been provided. In other words, an additional factor of safety of about 50 per cent has been provided, over the allowance made in estimating maximum rate of flow.

Where main sewers lie deep and the branches discharging into them would naturally be much higher, well holes are sometimes used to connect the two. These devices are described in Chap. XV, which also gives a description of flight sewers, occasionally required where a heavy drop in the grade of a main sewer is necessary.

A feature of design which should be mentioned in this place was stated as follows in Hering's report to the National Board of Health in 1881:

> The junction angle of converging sewers should be arranged so that the direction of flow of the two streams before joining is as nearly as practicable the same. Neither will then lose much velocity in endeavoring to overcome the change in direction. The less the sizes of the respective streams differ from each other, the more essential is this consideration. An important feature of junctions is the relative height of the joining streams, for unless this point is considered, backwater and deposits may occur in one of them. Theoretically, the joining sewers should be so shaped as to constantly deliver the sewage of each at the same level. To comply with this demand on all occasions is impossible, and it will suffice to consider the ordinary flow which occurs during nine-tenths of the time. The surface of the latter in the branches should be either the same for all, or increase in height as the bulk of the sewage becomes less. In other words, the smaller sewers should join the larger ones so that their ordinary flows meet at the same level, or so that the smaller sewer discharges at a higher level. When two sewers discharge into a manhole opposite to each other, at points above its bottom, they should be placed at different heights, or else receive a slight lateral turn, so that the full discharges do not directly meet each other.

Main sewers on the combined system are such expensive works that every opportunity should be sought for reducing their cost legitimately. Sometimes this can be done by providing several outlets where surplus storm flow can escape through short channels or conduits to neighboring bodies of water, and at London provision has even been made to pump some of this storm flow into the Thames rather than give the

long main sewers the size needed to handle it. These pumping stations are operated by gas engines, and are run only when the storm flow must be handled. Sometimes the first cost of combined main sewers can profitably be reduced, where the cost of construction is not heavy, by employing a rather small cross-section and constructing another trunk sewer later when it is needed. Where construction is expensive on account of poor ground or the presence of large amounts of water, or imposes a serious burden on the business of the streets in which it is carried on, it is usually advisable to design the main sewers to serve the community for the entire period which the interest rates on the cost of the sewers make most economical. Often the most satisfactory method of keeping down the cost of combined trunk sewers is to run them to the nearest bodies of water and draw off the dry-weather sewage into intercepting sewers near their lower ends. The extreme lower ends of the main sewers thus discharge storm water during rains while at other times the house sewage passes into the intercepting sewers. The methods of delivering the sewage into the intercepting sewers are explained in Chap. XVII.

Intercepting sewers, or collectors, are of two distinct types. The first receives part or all of the sewage of the system above a given contour, and is employed either to permit a reduction in the size of the intercepters at lower levels or to discharge by gravity the sewage from districts high enough to make pumping unnecessary. In the latter case, low-level main sewers are employed to convey the sewage from the lowlands to the pumping stations. The second type of intercepter crosses the lower ends of the main sewers of combined systems and receives the dry-weather sewage carried by them. By restricting its duty mainly to the house sewage, it can be kept of relatively small size and the sewage can thus be conducted by it in the most economical manner to the place of disposal. By a suitable allowance in the design of the special structures for intercepting the house sewage, the offensive first wash of the storm may also be diverted, if necessary.

Intercepters are given capacities determined by the methods explained in Chap. V. They generally are designed to carry a quantity equivalent to from 300 to 400 gal. per person from a population estimated to exist from 30 to 40 years after the date of the designs.

Relief sewers are built to take part of the sewage from a district where the main or intercepting sewers are already overcharged or are in danger of becoming so. They may be used to take excess storm water where it threatens to surcharge old sewers, as happens when the area of impervious land increases greatly or additional territory is drained into these old main lines, or they may be made to serve constantly a given district and be connected with the sub-mains and laterals in it, so as to restrict the service of the older main sewers to a more distant district. Experi-

ence in large cities, notably in London, shows that more than one relief sewer may eventually become necessary for a given district.[1]

The need for relief sewers is not necessarily an indication of any error in the original plans of a sewerage system. As already stated, it may be wise under some local conditions to use rather small main sewers at first, particularly if there is considerable doubt as to the direction in which the city's population will extend. If funds permit and the difficulties of construction are great, it is better as a rule to provide generous capacity at the outset.

Outfall sewers are the large lines leading from the lower ends of the collecting system to the places of disposal. The end of an outfall sewer running into water is termed its outlet; "outfall" is sometimes used for "outlet." The discharge of a sewer which is partly or wholly submerged is discussed in the next chapter.

Inverted siphons are sewers in which sewage runs under pressure due to their dropping below the hydraulic grade line and then rising again. The name is a poor one, but not so bad as "siphons," which is occasionally employed, although that term means something entirely different. It would be much better to speak of all such sewers as pressure or depressed sewers. They are most frequently employed to cross under rivers, but occasionally are needed on outfalls to avoid the long lines which would be required to keep the sewers on the hydraulic gradient, or to make pumping unnecessary.

In their design it is necessary to allow for internal pressure, and until recently cast-iron or steel pipe has generally been employed for them. With the development of reinforced concrete, however, a new material has become available for pressure sewers built in the trench, which have been constructed of noteworthy dimensions in Paris, and still more recently reinforced-concrete pipes of large size have been made and used successfully for carrying water under pressures up to 90 lb. per square inch.

The various details at the ends of inverted siphons are described in Chap. XVI. In any case where such siphons are employed, care should be taken to provide blowoffs at the lowest points, if possible, and to prevent, so far as practicable, coarse materials from entering them. In

[1] The admission of the runoff from roofs into separate sewers may prove the cause of an early necessity for greater sewerage facilities in a district where this practice is permitted. For example, in some of the sections of Cincinnati, which were provided with separate sewers before they were annexed, the runoff from roofs was permitted to be discharged into the sewers. After these sections of the city became built up to a greater extent, this runoff overtaxed the sewer capacity, and storm sewers were added as street improvements were made. In cases like this, relief sewers of districts provided with separate sewerage systems become absolutely necessary and it not infrequently happens that the existing separate sewers are retained for that purpose exclusively and combined sewers of large size are built to act as main sewers receiving the discharge from the separate sewerage system and also from storm-water drains.

connection with such a pressure sewer at Fitchburg, Mass., for example, a grit chamber with screens was provided, and a blowoff branch has been built to the Nashua River.

Force mains are pressure sewers through which sewage is pumped. Where small pumping stations are used to avoid placing sewers in deep trenches, it is often desirable to concentrate the lift at the stations, the sewage flowing to them by gravity and, after being lifted, flowing away by gravity, thus avoiding the use of long force mains.

Flushing sewers are occasionally used in sewerage work to flush out water courses receiving sewage or to convey water for flushing to the head of the lines to be kept clean. They are not sewers, strictly speaking, but water conduits. Milwaukee, Chicago, and Brooklyn possess flushing works of the first class. A good example of the second class was proposed by James H. Fuertes, in 1912, for use in connection with new sewers at Hoboken, N. J. This plan called for large, shallow, reinforced-concrete tanks at the heads of the flat trunk sewers needing flushing. The tanks were to be supplied with harbor water through pipe-flushing sewers built into the concrete foundations of the main sewers, a flap valve being placed on the end of the supply pipe in each tank. In this way, the tanks would be filled on rising tides and the flap valves would prevent the escape of the water as the tide falls. At the proper time on the falling tide, a sluice gate would be opened automatically and quickly to let the water run out of the tank into the sewer, the operation of the gate being controlled by a float.

GENERAL DETAILS OF SEWERAGE SYSTEMS[1]

Grades.—Although the grade of the invert is usually meant when the grade of a sewer is mentioned, in determining the cross-sections of combined and storm-water sewers the surface of the flowing sewage or the hydraulic gradient must be the controlling grade. In the case of separate sewers for house sewage alone, this distinction is rarely important and consequently is generally disregarded, but with combined sewers, where the surface of the water in the sewer during heavy rains may have a smaller slope than the invert, the surface gradient must be the controlling inclination or unpleasant conditions may arise like those which existed in Brooklyn, as mentioned in the Introduction.

The invert grade is the most important factor controlling the flow in separate sewers, and in combined sewers while only the dry-weather sewage is flowing. As explained in detail in the next chapter, the slope, S, is commonly taken as equal to v^2/C^2R, where v is the velocity, C is an

[1] In this subchapter, the authors have adopted many of the methods of presenting the subject which are found in Frühling's "Die Entwässerung der Städte," fourth edition, 1910, a treatise embodying the results of the investigations and studies of one of the foremost German sewerage specialists.

empirical coefficient, and R is the hydraulic mean radius or the area of the cross-section of the flowing stream divided by the length of the portion of the perimeter of the section which the water touches. The applicability of the formula at low depth of flow is questionable, and such data as are available indicate that when the depth is less than 2 in. or thereabouts, the velocities are much less than those obtained by the formula. This results in the stranding of suspended matter on the invert until it is flushed out by a larger flow than usual. In the case of the smallest laterals, it is inevitable for them to be dry near their dead ends at times, and a mere trickle of sewage generally flows through them, so that the stranding of suspended matter in them is common and they are often kept clean by flushing, either by hand or by automatic apparatus described in Chap. XVI. As a result of experience and observation, American sewerage specialists have reached a fairly uniform practice in respect to minimum grades for these small sewers, which is explained in detail in Chap. III. A rule for the minimum grade much used in England is to make it equal to $\frac{1}{5d + 50}$, where d is the diameter in inches. In Germany, circular house connections with a diameter of 4 to 5 in. are given slopes of 1:15 to 1:30, if possible; house connections of 6-in. diameter, slopes of 1:20 to 1:50; lateral sewers up to 12-in. diameter, slopes of 1:30 to 1:150, and from 12- to 24-in. diameter, slopes of 1:50 to 1:200. With egg-shaped sections, the minimum slopes are somewhat reduced; the preferred range of grade of branch sewers of such a section is from 1:100 to 1:300. In large main sewers the grades can be still further reduced, as explained in detail in Chap. III.

It is not always practicable to adhere to the standard minimum grades, for flat topography, a high level of the ground water or the necessity of pumping the sewage may render it advisable to reduce the slopes. The absolute minimum for laterals in Germany is about 1:250 for sizes up to 12 in. and about 1:400 for those between 12 and 24 in., while the sub-mains may sometimes be as flat as 1:1,000; in the United States, the practice is to establish certain minimum grades (see Chap. III) for use in the drafting room, and, if still lower grades appear necessary, to have the matter submitted for decision to the chief engineer's office. It is probable that the larger quantity of sewage resulting from the more liberal use of water in the United States accounts, in part, for the adoption of flatter grades for small sewers here than in Germany.

The maximum limit for grades has been less discussed in the United States than has the minimum limit, but it is an important matter, particularly with combined sewers and storm-water drains, where high velocities of discharge may cause the suspended sediment to erode the inverts and walls. In Germany, the maximum for small house drains is about 1:10, for 6-in. house drains about 1:15; for laterals about 1:20.

The disadvantage of steep slopes in small sewers is in the probability that the water will flow off so rapidly that the large floating matter will become stranded on the invert and will not be dislodged by the next wave passing down the sewer. In small sewers, it is practicable to avoid these steep grades by using drop manholes, and on sub-mains and

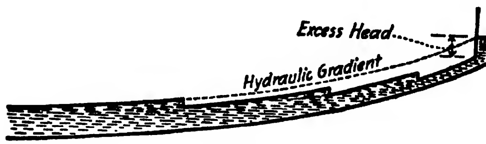


Fig. 5.

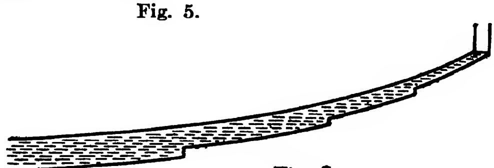


Fig. 6.

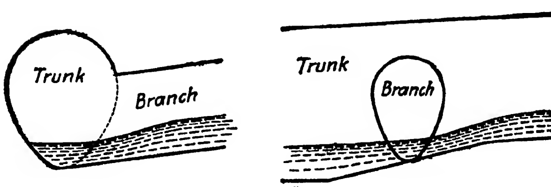


Fig. 7.

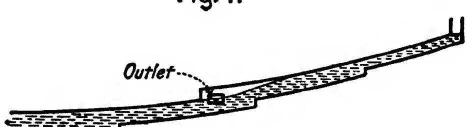


Fig. 8.

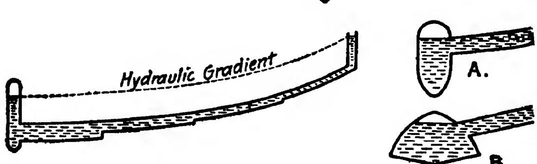


Fig. 9.

Figs. 5 to 9.

mains by using flight sewers; these special details are described in Chap. XV.

Turning aside from these grade relations, the profile of the invert of a long sewer is frequently a concave curve with the steeper part at the smaller end. If it is desired to have the hydraulic grade lie parallel with the invert and at the same time have the sewer run full, it follows

that a part of the sewer must be under pressure during storms, as shown in Fig. 5, the amount of the pressure being determined by the position of the hydraulic gradient. If it is desired to avoid this, the invert slope must be increased to provide the additional head, which will result in some sections of the sewer running only partly full, or the invert must be stepped from time to time (Fig. 6), or the cross-section must be widened. Stepping the invert involves laying the upper portions of the sewer at a lesser depth, but it can be arranged to give better details than with the continuous invert if the drops in grade are located at the inlets of the larger branches, as shown in Fig. 7. Such a detail avoids a reduction in velocity in both the sub-main and main sewer, which is particularly desirable with low depths of sewage when solids are most likely to drop to the invert. On the other hand, the upper portions of the sewer may not be deep enough to properly serve deep basements, unless the depth of the whole sewer is increased. As already mentioned, it is practicable to avoid checking the velocity in the branch by giving the latter a suitable elevation above the invert of the main into which it discharges, but this arrangement does not help the unfavorable condition in the main sewer.

A special condition arises in combined sewers where there is a relief outlet. When a large amount of storm water is flowing and the outlet is in operation (Fig. 8), there is an increase in the hydraulic gradient for some distance above the outlet. Moreover, in the part of the sewer affected by this change in the hydraulic gradient, the entering branches are also similarly affected and there is a corresponding general increase in velocity. This fact is rarely taken into consideration, nor an unfavorable consequence of it if the sewers are not kept clean, *viz.*, the picking up and sweeping along of sludge previously deposited above the outlet, and its discharge through this outlet.

A frequent cause of congestion in a branch sewer, and its attendant surcharge, is indicated in Fig. 9, where an increase in the elevation of the branch and its tributary lines is impracticable on account of the local conditions. The surcharge of the branch can be avoided in this case by lowering the trunk sewer, as in *A*, or it can be at least greatly reduced by employing a wider section, as in *B*, which will lower the hydraulic gradient. It is also possible to give the branch sewer a larger section, high and narrow, and thus reduce its hydraulic gradient, but this is expensive and it may prove best to build the lower stretch of the branch very carefully with the object of allowing it to carry internal pressure at times.

In laying out a combined sewerage system it is evident, from what has been said, that it is usually first necessary to determine the minimum permissible elevation of the sill of the lowest relief outlet. This will enable the elevation of the trunk sewer at that point to be established,

and from that elevation the grades of the upper portion of the system can be worked out. The best location of the various lines can only be determined by a number of trials, in many cases, and the failure to give proper study to grades and hydraulic gradients has been the cause of much of the unsatisfactory service of sewerage systems. The work is not unlike, in some respects, the location studies of railway lines, which have also frequently been hurried along, to the great disadvantage of the subsequent operation of the roads.

Relief Outlets.—Relief outlets for the escape of storm flow from large sewers into nearby rivers or lakes are an essential feature of most systems of combined sewers, for otherwise the trunk sewers would require enormous dimensions. In rare cases, as in New Orleans, it is necessary to collect and pump all the storm water, and under such conditions a separate system with independent drains for removing the rainfall may be the best solution of the sewerage problem. The treatment of all the rain-water of a city has rarely been considered necessary, and the usual problem in design of combined sewers is to determine what dilution of the house sewage with rain water is desirable before the mixture may be discharged through the relief outlets.

There will be some sewage escape into the river or lake whenever there is a discharge through one of these storm overflows. If the sewers are not kept clean, the amount of organic matter which is discharged in this manner will be higher than otherwise, because the scouring action of the storm water in the sewers will sweep it from the inverts where it has settled during dry weather. But, as many rainfalls will not yield enough water to bring the storm overflow into service, although they will increase the flow in the sewers enough to take up some of the deposits on the inverts, it is apparent that with well-designed and well-built sewers, the uncertainty as to the degree of dilution of the house sewage during heavy storms will be unimportant in most cases. The relief outlets do not usually discharge often enough in a well-designed system to make the amount of organic matter escaping through them into the river of significance as respects the condition of the latter.

There has been a wide variation in the ratio of storm water to house sewage adopted as the basis for the design of the relief outlets. It is naturally larger when the outlet discharges into a small sluggish stream than where there is a larger body of water to receive the excess quantity. If the outlets are along a river and it is more desirable to keep its upper course uncontaminated than its lower course, the storm overflows along the latter should be much larger than the others, even though this makes it necessary to employ larger main sewers than would otherwise be necessary between the first and last points of relief.

The value of the ratio has ranged from about 2 to 8. The phenomena that take place in a sewer during the period when the overflow is in

service have not been investigated so fully as is desirable. As already explained, there is an increased velocity of flow in the sewer when the overflow is discharging, and this results in a somewhat larger volume of sewage continuing in the trunk sewer than would be the case if the slope were that of the invert. Furthermore, the discharge of a weir parallel to the thread of the current is not so great as when the weir is at right angles to the current.

Numerous relief outlets have the dual advantage of keeping down the size of the sewers and discharging the excess storm water at several places rather than concentrating it at one. The cost of the outlet conduits from the overflows to the points of discharge, as compared with the cost of sewers of different sections, will afford a useful guide to the best number. Old sewers and the channels of brooks can sometimes be utilized to advantage as outlet conduits.

The discharge over the sill of a relief outlet depends on the elevation and length of sill, the head on the weir, or depth of flow in the sewer, the shape of the outlet, the dimensions of the main sewer above and below the outlet and possibly other factors. Some experimental information is available on the discharge of side weirs and is discussed in greater detail in Chap. XVII.

It is not known how closely these experimental results correspond with discharges of larger structures but until more data are available, they may serve as an approximate basis for estimates. For the range of weir lengths investigated in the experiments, see page 632.

If the relief conduit is so designed that its lower end is completely closed by high water in the river or lake into which it discharges, the hydraulic gradient of the conduit should be investigated to make sure that backing-up of the water in the conduit will not interfere with the free action of the weir.

Below the relief outlet the main sewer carries a smaller quantity of sewage than above it, and, with the same grade, it may be given a smaller cross-section. With an increase in elevation of the sill of the overflow there is an increase in the quantity of water which remains in the main sewer. A long sill is better than a short one for regulating the quantity of water which escapes and, consequently, the quantity which remains in the sewer.

If, for any reason, the sill of the storm overflow must be placed so low that the floods in the river rise above it, but not to the crown of the trunk sewer, the discharge of the overflow will then be checked. There are no published observations of what the discharge will be under such conditions, but from Herschel's discussion of the flow over submerged weirs[1] and adopting only two-thirds of his quantities, the volume of

[1] *Trans. Am. Soc. C. E.*, **14**, 194.

sewage escaping from an overflow under such conditions will probably not fall below an amount given by the expression $Q = nlH^{3/2}$, where Q is the discharge in cubic feet per second, l is the length of the sill, H is the depth of the sill below the water surface in the sewer, and n is a coefficient taken from the following list and depending upon the ratio of h, the height of water in the relief channel above the sill, to H.

h/H....	0.1	0.2	0.3	0.4	0 5	0 6	0.7
n.......	2 2	2.2	2.1	2.0	1 6	1 3	1 1

Preliminary Studies.—In making the preliminary studies of a system of sewers, it is possible to use merely tables of the discharge of sewers laid on a grade of 1 per cent. Table 2 is an example of such a table, based on a value of $n = 0.013$ in the Kutter formula, explained in the next chapter. Some engineers prefer to use such tables and a slide rule to reading quantities from diagrams like those given in the next chapter, and to illustrate their use as well as to introduce at this point some of the more general problems arising in sewerage work, a few examples of preliminary studies (adapted from Frühling's "Entwässerung") are given here. The basic fact to be kept in mind is that velocities and discharges vary about as the square roots of the slopes.

1. A sewer 1,476 ft. long with a fall of 6.56 ft. must discharge 4.097 cu. ft. per second; what should be its diameter and velocity?

The average slope is 6.56:1,476 = 1:225. The tables are prepared for slopes of 1:100; velocities and discharges for other slopes vary as the square roots of the slopes. The discharge on a slope of 1:100 corresponding to 4.097 c.f.s. on 1:225 is $4.097\sqrt{(225/100)}$, which is readily found by slide rule to be 6.15 c.f.s. If it is desired to have the sewer run full when discharging, Table 2 indicates that a 15-in. circular section will be correct, and the velocity will be about 3½ ft. per second. The velocity with smaller discharges may be found by dividing the tabular velocities for the different depths of sewage by $\sqrt{(225/100)}$. It is evident that the velocity sinks to 2½ ft. when the sewage has a depth of less than about 5 in.

2. The 15-in. sewer of Ex. 1 discharges 0.053 cu. ft. per second during dry weather into a 54-in. sewer on a grade of 1:1,200, carrying 0.88 c.f.s. of house sewage; what is the best way to prevent backing-up of sewage at the junction?

The corresponding discharge of the sub-main with a slope of 1:100 will be $0.88\sqrt{(1{,}200/100)}$ or 3.05 c.f.s., which Table 2 shows will fill less than 0.1 of the depth of the section or, say, 5 in. In the same way, the depth of flow in the 15-in. sewer with 0.053 c.f.s. may be found to be less than 0.1 of its diameter, or, say, 1½ in. Hence, as a first approximation, it may be assumed that the invert of the lateral must be 5 − 1½ = 3½ in. above the invert of the main branch to cause the surface of the sewage in the two sewers to be at the same elevation. This results in a flattening in the invert grade in the lateral, which is not likely to be of importance except where the

available fall or slope is restricted. The discharge of 0.053 into 0.88 c.f.s. will cause only a trifling increase in depth and loss of velocity in the main sewer. After the general layout has been worked up approximately, the elevation of the sub-main sewer at the junction may be readjusted by the more accurate methods explained in the next chapter.

3. The sub-main sewer of Exs. 1 and 2 is assumed to be two-thirds filled; what will be the effect of this condition on the lateral?

Two-thirds of 54 in. is 36 in., which represents the depth of sewage in the sub-main. If the position of the lateral is fixed, as in Ex. 1, this sewer will be under an internal pressure of 17½ in. at the crown, at its junction with the sub-main. If the total available fall from the invert of the lateral at its upper end to the invert of the sub-main is 6.56 ft., and it is not to be under pressure at the upper end, the hydraulic grade, when flowing full, will be $\frac{6.56 + 1.25 - 3.00}{1{,}476} = \frac{1}{307}$, and the capacity of the lateral will be about 3.65 cu. ft. per second.

4. In the 54-in. sewer, 2,100 ft. below the junction of the 15-in. lateral, there is a relief outlet which draws the water down to a depth of 27 in. during a storm. Will its effect be felt at the junction point, and if so, approximately how much will it lower the water at that point?

This problem may be solved approximately by dividing the length into several sections short enough so that conditions of uniform flow may be assumed to exist in each section. For instance, assume 14 sections of 150 ft. each. The starting point is the elevation of the water opposite the outlet, or 27 in. above the invert. The sewer flowing full has a capacity of $202\sqrt{100/1{,}200} = 58.4$ cu. ft. per second; assume this to be the total flow. At the outlet this quantity is flowing in a section 27 in. deep or 50 per cent of the depth. The ratios of velocities and discharges at any depth to those at full depth, as will be explained in Chap. III, are about as follows:

Proportionate depth	Proportionate velocity	Proportionate discharge
0.1	0.38	0.02
0.2	0.61	0.09
0.3	0.77	0.20
0.4	0.90	0.34
0.5	1.00	0.50
0.6	1.09	0.67
0.7	1.12	0.84
0.8	1.14	0.97
0.9	1.13	1.06
1.0	1.00	1.00

At 50 per cent of the depth the sewer is carrying 50 per cent of its capacity full with the same hydraulic grade. Then 58.4/0.50 = 116.8 cu. ft. per second is the corresponding capacity of the full section, and the hydraulic grade near

the outlet is $\frac{116.8^2}{202^2}\left(\frac{1}{100}\right) = 1{:}300$. Following this method, the depths of flow at the successive points may be estimated thus:

	Inches
Outlet	27
Point 1, $27 + \left(\frac{1}{360} - \frac{1}{1{,}200}\right) 150 \times 12 =$	31.5
Point 2, $31.5 + \left(\frac{1}{490} - \frac{1}{1{,}200}\right) 150 \times 12 =$	33.7
Point 3, $33.7 + \left(\frac{1}{600} - \frac{1}{1{,}200}\right) 150 \times 12 =$	35.2
Point 4, $35.2 + \left(\frac{1}{680} - \frac{1}{1{,}200}\right) 150 \times 12 =$	36.4
Point 5, $36.4 + \left(\frac{1}{750} - \frac{1}{1{,}200}\right) 150 \times 12 =$	37.3
Point 6, $37.3 + \left(\frac{1}{810} - \frac{1}{1{,}200}\right) 150 \times 12 =$	38.0
Point 7, $38.0 + \left(\frac{1}{860} - \frac{1}{1{,}200}\right) 150 \times 12 =$	38.6

The surface curve is now so flat that, for this approximate computation, the increase in depth may be taken as ½ in. per 150 ft. for the remainder of the section, or 3½ in. more to the junction point, where the total depth will be about 42 in.

5. A flow of 44.15 cu. ft. per second must be carried by a sewer with a grade of 1:900. Conditions make it necessary that sewage shall not be

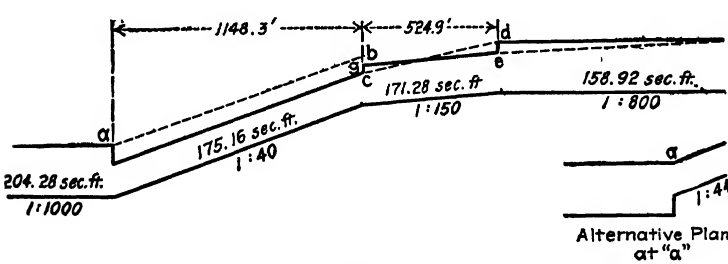


FIG. 10.

backed up to a depth greater than 36 in. above the invert. What section should be selected?

A discharge of 44.15 on 1:900 grade corresponds to $44.15\sqrt{900/100} = 132.45$ on 1:100 grade. A 45-in. sewer at 0.9 depth has this capacity, but $0.9 \times 45 = 40\frac{1}{2}$ in. is the corresponding depth of flow, which is too high. A 48-in. sewer on 1:100 slope has a capacity of 132.45 at about 0.74 of the depth or 35½ in., and therefore fulfils the conditions.

6. Owing to topographical conditions, a main sewer must have the profile shown in Fig. 10. What are the cross-sections and hydraulic gradients for the given invert grades and quantities?

The computation begins with the lowest stretch of sewer. The equivalent discharge on a 1:100 grade is $204.82\sqrt{(1{,}000/100)} = 648$ c.f.s., an amount so large that an aqueduct section of the semielliptic, semiparabolic, segmental, horseshoe or other type, described in Chap. XII, will be most advantageous.

If the next stretch were to run full with the quantity stated on the profile, it would operate under the head due to the hydraulic gradient *ab*, or at least with the gradient *ac*. In view of the abundant grade, the alternative arrangement at *a*, with a drop of some sort, such as a flight sewer or well hole, is preferable. The invert and hydraulic gradients are determined by trial, assuming for a first approximation a 60-in. section and that the lowest stretch is a semielliptical section 90 in. high. Then the slope of the crown, which is assumed to coincide with the hydraulic gradient *ac*, will be

$$\frac{1}{1{,}148.3}\left(\frac{1{,}148.3}{40} - \frac{90}{12} + \frac{60}{12}\right) = \frac{1}{43.8}$$

The corresponding volume of water on a 1:100 grade will be $175.16\sqrt{(43.8/100)}$ or 116 sec.-ft., which requires a sewer 45 in. in diameter. The assumption of a 60-in. sewer is therefore in error. Changing to a 45-in. section, the slope of the crown is found to be $\frac{1}{46}$; the corresponding capacity for 1:100 grade will be 119 cu. ft. per second, which is provided by the 45-in. sewer.

Either *ge* or *cd* may be taken for the hydraulic gradient of the next stretch. If the former is assumed, there will be no internal pressure upon the crown of this stretch, but there will be some pressure at the upper end of the stretch below, and its hydraulic slope and capacity will be slightly reduced. Whether the internal pressure *de* would be objectionable will depend on its magnitude, and it is therefore desirable to estimate approximately the cross-section of the stretch *ef* before fixing that of *be*. The required capacity of 158.92 c.f.s. on a slope of 1:800 corresponds to 450 c.f.s. at 1:100. This quantity is beyond the scope of Table 2, and a sewer larger than 60 in. diameter will be required. By reference to Fig. 16 it is seen that a 72-in. sewer has the required capacity. The available slope from *c* to *d* is then about 5.75/524.9 or 1:91. The required capacity of 171.28 c.f.s. at this slope corresponds to 163 c.f.s. at a slope of 1:100, for which a 51-in. sewer is adequate. Making the computation as though the entire section were filled for the whole length, a 51-in. sewer carrying 171.28 c.f.s. would have a hydraulic gradient of 1:97, and the internal pressure at the crown of the sewer at the point *e* would be

$$\frac{524.9}{97} + \frac{45}{12} - \left(\frac{524.9}{150} + \frac{51}{12}\right) = 1.41 \text{ ft.} = 17 \text{ in.}$$

If it is desired to avoid this internal pressure, the stretch must be designed on the basis of the invert grade, in which case the discharge on a 1:100 grade is $171.28\sqrt{(150/100)} = 211$ c.f.s., calling for a 54-in. sewer. In this case there will be a lowering of the surface of the sewage in the top stretch of sewer, as shown in the illustration.

7. A storm-water overflow is located as shown in Fig. 11; what is the length of its sill if the overflow is assumed to come into operation on a fivefold dilution of the dry weather sewage?

TABLE 2.—VELOCITY IN FEET PER SECOND (v) AND DISCHARGE IN CUBIC FEET PER SECOND (Q) OF CIRCULAR SEWERS WITH DIFFERENT PROPORTIONS (H) OF THE DIAMETER (D, INCHES) FILLED AND A GRADE OF 1:100

Based on n = 0.013 in Kutter's Formula

D/H	4 in.		6 in.		8 in.		10 in.		12 in.		15 in.		18 in.		20 in.		22 in.		24 in.		27 in.	
	v	Q	v	Q	v	Q	v	Q	v	Q	v	Q	v	Q	v	Q	v	Q	v	Q	v	Q
0.1	0.58	0.0026	0.81	0.009	1.04	0.019	1.26	0.036	1.45	0.059	1.76	0.11	2.04	0.187	2.23	0.25	2.40	0.33	2.56	0.42	2.82	0.584
0.2	0.98	0.0121	1.40	0.041	1.775	0.0879	2.12	0.164	2.47	0.276	2.93	0.51	3.36	0.848	3.74	1.16	3.98	1.49	4.23	1.89	4.63	2.62
0.3	1.34	0.0294	1.87	0.093	2.37	0.208	2.84	0.375	3.24	0.645	3.83	1.19	4.45	1.98	4.83	2.65	5.15	3.43	5.50	4.36	6.01	6.03
0.4	1.59	0.0518	2.25	0.164	2.82	0.369	3.33	0.678	3.84	1.13	4.60	2.12	5.31	3.50	5.71	4.66	6.12	6.02	6.48	7.62	7.08	10.53
0.5	1.81	0.0795	2.54	0.249	3.19	0.558	3.79	1.03	4.35	1.71	5.17	3.17	5.90	5.22	6.40	7.01	6.84	9.02	7.28	11.45	7.90	15.78
0.6	1.97	0.108	2.76	0.339	3.44	0.755	4.09	1.40	4.69	2.31	5.60	4.31	6.36	7.04	6.90	9.46	7.38	12.18	7.83	15.42	8.49	21.15
0.7	2.10	0.1365	2.91	0.431	3.60	0.943	4.32	1.76	4.95	2.91	5.87	5.39	6.66	8.90	7.24	11.80	7.70	15.18	8.18	19.20	8.89	26.40
0.8	2.12	0.159	2.96	0.498	3.72	1.115	4.42	2.06	5.01	3.37	5.97	6.30	6.78	10.28	7.34	13.72	7.81	17.65	8.44	22.40	9.03	30.78
0.9	2.11	0.175	2.92	0.563	3.64	1.21	4.34	2.24	4.96	3.70	5.90	6.87	6.69	11.20	7.26	15.03	7.72	19.30	8.22	24.50	8.92	33.65
1.0	1.81	0.158	2.54	0.498	3.19	1.115	3.79	2.06	4.35	3.42	5.17	6.36	5.90	10.45	6.39	14.00	6.83	18.03	7.28	22.90	7.90	31.55

D/H	30 in.		33 in.		36 in.		39 in.		42 in.		45 in.		48 in.		51 in.		54 in.		57 in.		60 in.	
	v	Q	v	Q	v	Q	v	Q	v	Q	v	Q	v	Q	v	Q	v	Q	v	Q	v	Q
0.1	3.07	0.77	3.31	1.02	3.52	1.30	3.75	1.62	3.96	1.99	4.20	2.41	4.45	2.89	4.63	3.41	4.81	3.98	5.02	4.63	5.22	5.32
0.2	5.01	3.51	5.38	4.58	5.78	5.84	6.11	7.22	6.44	8.82	6.77	10.65	7.10	12.72	7.40	14.95	7.69	17.38	8.02	20.40	8.26	23.15
0.3	6.47	8.03	6.96	10.45	7.42	13.20	7.81	16.37	8.23	20.00	8.65	24.12	9.06	28.70	9.46	33.85	9.82	39.45	10.20	45.55	10.60	52.50
0.4	7.61	13.93	8.14	18.05	8.67	22.90	9.18	28.45	9.66	34.70	10.14	41.85	10.59	49.70	11.06	58.60	11.50	68.25	11.90	78.60	12.31	90.4
0.5	8.50	20.80	9.08	26.95	9.66	34.05	10.23	42.45	10.76	51.80	11.19	61.70	11.75	73.85	12.26	87.00	12.73	101.2	13.16	116.5	13.68	134.3
0.6	9.12	28.10	9.78	36.40	10.39	46.00	10.97	57.10	11.54	69.40	12.10	83.60	12.65	99.50	13.15	116.70	13.63	135.8	14.05	156.0	14.64	180.0
0.7	9.55	35.05	10.23	45.30	10.84	57.25	11.48	71.25	12.06	86.70	12.66	104.6	13.16	123.7	13.74	145.70	14.26	169.6	14.76	195.6	15.26	224.0
0.8	9.68	40.80	10.40	52.90	11.04	66.85	11.68	83.10	12.28	101.2	12.88	122.1	13.40	144.0	13.99	170.0	14.50	197.5	15.02	228.0	15.53	261.5
0.9	9.58	44.55	10.28	57.80	10.87	72.80	11.50	90.45	12.10	110.3	12.68	132.8	13.23	157.3	13.79	185.0	14.32	215.8	14.82	249.0	15.33	285.0
1.0	8.50	41.70	9.08	53.90	9.66	68.10	10.23	85.00	10.75	103.5	11.19	123.6	11.75	147.8	12.26	174.0	12.73	202.5	13.16	232.8	13.68	268.5

The quantities of sewage and the invert grades are indicated in Fig. 11; the numbers in parentheses are the quantities during heavy storms, while the smaller numbers are the quantities flowing when the dry-weather sewage has received a fivefold dilution. Sewer V has to carry, before the relief outlet comes into action, $4.13 + 2.90 + 3.99 = 11.02$ cu. ft. per second, consisting of 1.84 cu. ft. of dry-weather sewage and 9.18 cu. ft. of storm water. This quantity on a 1:140 slope corresponds to 13.03 c.f.s. on a 1:100 grade. If the hydraulic gradient is assumed provisionally to be parallel to the invert, the sewer will need a section 20 in. in diameter. Since it is not feasible to draw the sewage down flush with the weir crest at its lower end for reasons that will appear later, sewer V will carry more than 11.02 c.f.s. at maximum discharge and a diameter of 27 in. is assumed. The 11.02 c.f.s. discharge, or the corresponding 13.03 c.f.s. on a 1:100 grade, fills the 27-in. section to a depth of about 12 in., which determines the elevation of the sill of the relief outlet.

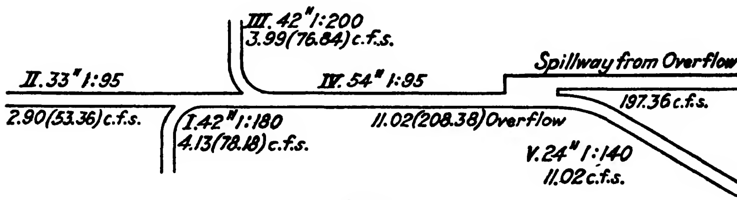


Fig. 11.

Referring to Babbitt's formula for overflow weirs as given on page 632, it is evident that to reduce h_2, the head on the weir at its downstream end, to zero, thus bypassing all of the flow in excess of 11.02 c.f.s., an infinite length of weir would be required. To get an economical design a small head should be permitted at this point and a discharge greater than 11.02 will pass down sewer V for a brief period or while the overflow weir is in service. Assuming the inverts of sewers IV and V to be continuous, the head on the upper end of the weir, h_1, is $54 - 12 = 42$ in. or 3.5 ft. With $h_2 = 0.1$ ft. a length of weir of 209.0 ft. is required while if $h_2 = 0.25$ ft., this length is reduced to 155.0 ft. Increasing h_2 to 0.50 ft. decreases the length of weir to 114.2 ft.

Using Engel's formula, a weir crest of 10.5 ft. is obtained, but by substituting this value in Babbitt's formula, it is seen that h_2 is 2.92 ft. which will surcharge sewer V and greatly exceed the desired amount of 11.02 c.f.s. Babbitt's formula is in a more convenient form for conditions encountered in problems of this kind where h_2 must be closely controlled.

Allowing a value of 0.5 ft. for h_2 means that sewer V carries a depth of 18 in. or a discharge of 21.35 c.f.s. while with $h_2 = 0.25$ ft. a 15-in. depth is equivalent to a discharge of 16.0 c.f.s. and with $h_2 = 0.10$ ft. the discharge will be 12.82 c.f.s.

It is evident that the 27-in. pipe is larger than necessary. Refiguring for a 24-in. diameter, it may be seen that the sill of the weir is 13.5 in. above the invert with a length of 152.8 ft. discharging 192.45 c.f.s. while sewer V

takes 15.93 c.f.s. at a maximum depth of 16.5 in. or with $h_2 = 0.25$ ft. The 24-in. diameter will therefore be used for sewer V.

Sewer Sections.—The problem of the design of masonry sewers is not solved with the determination of the required carrying capacity, but includes a number of other features which may be of considerable importance.

The most economical shape for the waterway cross-section can only be selected after careful consideration of the special conditions imposed and the relative merits of one type as against another to meet these special conditions. While the circular cross-section has been used for a large number of the masonry sewers constructed in this country, there has been an increasing use of other forms such as the horseshoe, semi-elliptical, and rectangular sections. In the older combined sewerage systems constructed previous to 1890, and built for the most part of brick for sizes above 24 in. in diameter, the egg-shaped cross-section was frequently used, but since that time the extended use of concrete has caused it to decrease in popularity. The old Massachusetts North Metropolitan System was a departure from the practice of the time in that it included such types as the Gothic, catenary, and basket-handle sections. The general adoption of concrete for masonry sewers has brought about a more extended preference for the flatter types of inverts on account of their being more easily constructed than the inverts of circular or egg-shaped sections.

Aside from the hydraulic properties, such considerations as the method of construction, character of foundation, available space, and stability may be instrumental in determining the best type of sewer section to adopt for a given case.

The selection of the proper thickness of masonry for a given size of sewer, unless determined in the light of experience with similar structures, should be the result of a careful consideration of the forces to be encountered and an analysis of the stresses as determined by the best available methods. This applies particularly to the larger sewers, 6 ft. in diameter and over.

A study of existing sewers is one of the best guides to safe construction although not necessarily the most economical construction. Empirical formulas founded on experience have some value but should not be depended upon without an adequate analytical check.

The proper selection of the materials of construction involves not only a comparison of the cost of one material with that of another but also a consideration of the relative life and wearing qualities of the materials. The latter applies especially to the materials used for the lining of the invert.

In some localities, the erosion of sewer inverts has been a serious problem responsible for the failure of the entire structure. To resist

this wear, a lining of vitrified brick has been found satisfactory in some cases.

Sewers are subjected to the action of external forces due to surface loads transmitted through the backfill and to the pressure of the backfilling material itself. Surface loads may be divided into live and dead loads. The former include such loads as locomotives and other railroad rolling stock, road rollers, and heavy vehicles; the latter include loads from piles of lumber, brick, coal, and other materials commonly stored in commercial and manufacturing districts.

With the advent of reinforced concrete has come a greater need for the careful analysis of the masonry section for large sewers. With sewers constructed of brick or plain concrete, the sewer arch, if properly designed, is subjected only to compressive stresses and depends largely for its stability on the ability of the side walls or abutments to resist the arch thrust. With reinforced concrete, however, the structure as a whole from invert to crown can be designed to resist heavy bending moments and to act as a monolith.

The so-called "elastic theory" presents the most rational and practicable means for the analysis of sewer sections. The method of analysis under this theory as described by Turneaure and Maurer in "Principles of Reinforced Concrete Construction" is one of the simplest and best, but for an analysis of the structure as a whole, particularly where the sewer is to be built in compressible soil, the method developed by Prof. A. W. French for the authors is preferable.

Although the previously mentioned aids in design are of the greatest assistance, there must be behind them all sound judgment coming from experience if the best results are to be obtained.

DEPRECIATION OF SEWERS

A sewerage system represents the investment of a large amount of money, usually raised by issuing bonds. If municipalities paid as much attention to financial accounts as private corporations do, the present value of the sewerage and other public works would be ascertained from time to time, just as a railroad company revises its estimates of the value of its physical property. Or if sewerage works were often built by private corporations and taken over later by the municipalities, valuations of such works with accompanying estimates of depreciation would be made frequently enough to furnish significant information upon the progressive loss in value of such works with increasing age. It is probable that if annual charges for the use of the sewers were commonly made, the necessity for justifying such charges would lead to valuations and determinations of depreciation which would be of real significance. The fact is, however, that few if any systematic and

scientific attempts to determine the value and depreciation of sewers have been made, and such work as has been done along this line is not general enough to serve as a guide or to warrant any definite conclusions as to life of sewers and appurtenant structures.

A very large part of the sewers constructed prior to 1880, and some built since that date, were inadequate, and the methods and materials of construction employed in many of the early sewers were such that serious physical deterioration has occurred. Consequently, a compilation of data relating to the abandonment or reconstruction of old sewers would be of slight value in attempting to forecast the useful life of modern sewers built on adequate designs and of thoroughly resistant materials. Probably a careful inspection of the physical condition of the sewer structures, including tests of wear of inverts, condition of joints of brickwork and of pipe sewers, approximate determination of leakage into the sewers, wear of manhole frames and covers, and the like, would usually enable an experienced engineer to estimate fairly the "percentage condition" of the structures, and this combined with an allowance for functional depreciation resulting from changes in character of districts or annexation of territory, with consequent inadequacy, is likely to prove the fairest method of determining depreciation of sewers unless and until a sufficient body of data shall have been accumulated to serve as a guide in the determination of probable useful life and the mathematical computation of depreciation therefrom. Under any circumstances, it is likely that a thorough inspection will be necessary to an intelligent estimate of depreciation and future life.

The desirability of compiling information relating to actual useful life of sewers is obvious, and it is to be hoped that engineers having charge of sewer systems will put such data on record so that they may be summarized and available for use in the future.

CHAPTER II

HYDRAULICS: FLOW OF WATER AND SEWAGE

Hydrodynamics, Hydrostatics, Hydraulics.—The science of hydrodynamics is that branch of hydraulics which treats of the mechanics of fluids in motion. The science of hydrostatics, on the other hand, treats of the mechanics of fluids at rest.

The term hydraulics is here used as having the broader significance including both hydrostatics and hydrodynamics. This chapter, therefore, embraces a brief reference to water and some of its more important physical attributes, and to certain of the principles of hydrostatics and a more extended discussion of hydrodynamics or the principles governing flow, especially in sewers.

WATER AND SEWAGE

Water (H_2O) is a colorless liquid with high solvent powers. Having great fluidity, or little viscosity, it transmits pressures equally in all directions throughout its mass, the direction of the pressure being normal to the surface to which it is applied (Pascal's law).

Sewage is composed of about 99.9 per cent of water and but 0.1 per cent of mineral and organic matter,[1] and has a specific gravity but very little in excess of unity (1.001, approximately); it is treated in hydraulic discussions as if it were clear water. The retarding effects of its contents at times and under certain conditions, more particularly at the dead ends of the collecting system, are not to be lost sight of, however.

Compressibility.—In hydraulic computations, water may be assumed to be substantially incompressible, its coefficient of compressibility, or decrease in unit volume, caused by a pressure of one atmosphere (14.7 lb. per square inch), being approximately 0.00005. Its modulus of elasticity, E, in compression is approximately 296,000 lb. per square inch. The modulus increases and the coefficient of compressibility decreases slightly with increase in temperature. As an increase in pressure of 10 atmospheres increases the weight of water only by about 0.03 lb. per cubic foot, the effect of compressibility is negligible.

Molecular Changes.—Water reaches its maximum density at 39.3° F., at which point its specific gravity is unity. Water freezes at 32°

[1] This corresponds roughly to 1,000 parts per million of total solids.

F., at which temperature its specific gravity is 0.99987. It is owing to the fact that the maximum density of water occurs at a slightly higher temperature than the freezing point that bodies of fresh water do not freeze to a greater depth, for as the temperature of the water gradually falls in the early winter, the point of maximum density is reached at 39.3° F., and as the water chills further at the surface, by reason of its contact with the colder atmosphere, its specific gravity is raised and the cold layer of water therefore floats, except as wind currents may cause circulation and carry some of it to lower depths; and thus it continues to fall in temperature until the ice sheet forms.

Water boils at sea level (barometric pressure of 30 in. of mercury, or 34 ft. of water) at 212° F., when its specific gravity is approximately 0.95865.

Weight of Water, Ice, and Sewage.—Fresh water weighs about 62.43 lb. per cubic foot. For approximate computations, the unit 62.5 lb. is often used for convenience, and then

$$1 \text{ cu. ft.} = 62.5 \text{ lb.} = \frac{1{,}000}{16} \text{ lb.} = 1{,}000 \text{ oz.}$$

Salt water varies in density and weight, that of the Atlantic Ocean weighing, in the latitude of New York, approximately 64.1 lb., and in the Gulf of Mexico, 63.9 lb. The water in Great Salt Lake weighs from 69 to 76 lb. per cubic foot.

Ice weighs 57.2 to 57.5 lb. per cubic foot.

Sewage is usually assumed to have the same weight as water. In an investigation made by Harrison P. Eddy of the weight of the sewage discharged through the North Metropolitan Sewer at East Boston, a specific gravity of 1.0018 was found, the sewage having 1,022 parts of chlorine per million. This would correspond to an excess of 0.1 lb. per cubic foot, over the weight of fresh water, and this was a fairly strong American sewage, containing much salt or sea water.

The atmospheric pressure at sea level will sustain a column of mercury 30 in. high, in vacuum, and of water, 34 ft. As mercury weighs 0.49 lb. per cubic inch, this corresponds to $30 \times 0.49 = 14.7$ lb. per square inch pressure (1.033 kg. per square centimeter). This is known as the pressure of one atmosphere, a pressure of two atmospheres being double this amount, or approximately 29.4 lb. per square inch.

TABLE 3.—ATMOSPHERIC PRESSURES AND EQUIVALENTS
Merriman's "Treatise on Hydraulics," 1912.

Mercury barometer, inches	Pressure, pounds per square inch	Pressure, atmospheres	Water barometer, feet	Elevations, feet	Boiling point of water, degrees Fahrenheit
31	15.2	1.03	35.1	−890	213.9
30	14.7	1.00	34.0	0	212.2
29	14.2	0.97	32.9	+920	210.4
28	13.7	0.93	31.7	1,880	208.7
27	13.2	0.90	30.6	2,870	206.9
26	12.7	0.86	29.5	3,900	205.0
25	12.2	0.83	28.3	4,970	203.1
24	11.7	0.80	27.2	6,080	201.1
23	11.3	0.76	26.1	7,240	199.0
22	10.8	0.72	24.9	8,455	196.9
21	10.3	0.69	23.8	9,720	194.7
20	9.8	0.67	22.7	11,050	192.4

Gravity.—The acceleration due to gravity is approximately 32.16 ft. per second per second.

TABLE 4.—FUNCTIONS OF ACCELERATION DUE TO GRAVITY, g
Hughes and Safford's "Hydraulics."

	In feet		In meters	
	Number	Log	Number	Log
g	32.16	1.5073	9.803	0.9914
$2g$	64.32	1.8083	19.607	1.2924
$(2g)^{1/2}$	8.02	0.9042	4.428	0.6462
$\frac{2}{3}(2g)^{1/2}$	5.347	0.7281	2.952	0.4701
$\frac{1}{2g}$	0.01555	$\bar{2}.1917$	0.051	$\bar{2}.7076$

Various formulas for the value of g for any latitude and elevation have been devised. Hamilton Smith[1] quotes a formula used by the U. S. Coast and Geodetic Survey, and gives values of $(2g)^{1/2}$ based thereon, from which the following table has been prepared.

[1] "Hydraulics," 20.

TABLE 5.—VALUES OF g FOR VARIOUS LATITUDES AND ELEVATIONS

Latitude	Elevation in feet above sea level					
	0	1,000	2,000	3,000	4,000	5,000
0	32.089	32.086	32.083	32.080	32.077	32.074
20	32.109	32.106	32.103	32.100	32.097	32.094
40	32.159	32.156	32.153	32.150	32.147	32.144
60	32.216	32.213	32.210	32.207	32.204	32.201

Intensity of Water Pressure.—Ignoring the influence of changes in atmospheric conditions and external forces, the intensity of pressure on the unit of area, resulting from a column of fluid of given height, is equal to the weight of the fluid per unit volume, times its height.

$$P = wh$$

$$P \text{ in pounds per square foot } = 62.4h$$

$$P \text{ in pounds per square inch } = 0.4333h$$

$$h = P/w$$

$$h = 0.016P \text{ in pounds per square foot.}$$

$$h = 2.308\ p \text{ in pounds per square inch.}$$

Expressed in words, this means that a pressure of 1 lb. per square inch corresponds to a head of 2.308 ft. of water. A pressure of 1 kg. per square centimeter corresponds to a head of 10 m.

TABLE 6.—CONVERSION FACTORS FOR UNITS OF PRESSURE
Hughes and Safford's "Hydraulics," first edition.

	Feet of water	Log	Inches of mercury	Log	Pounds per square inch	Log	Pounds per square foot	Log
Pounds per square inch to..........	2.308	0.3632	2.037	0.3090	1.0000	0.0000	144.00	2.1584
Pounds per square foot to..........	0.01603	$\bar{2}$.2048	0.01414	$\bar{2}$.1506	0.00694	$\bar{3}$.8416	1.000	0.0000
Inches in height of mercury to......	1.133	0.0542	1.000	0.0000	0.4910	1.6910	70.699	1.8494
Feet in height of fresh water to....	1.000	0.0000	0.8826	$\bar{1}$.9458	0.4333	$\bar{1}$.6368	62.4	1.7952
Feet in height of sea water to.....	1.025	0.0107	0.9047	$\bar{1}$.9565	0.4442	$\bar{1}$.6475	64.0	1.8062
Atmospheres to....	33.923	1.5305	29.942	1.4763	14.70	1.1673	2116.8	3.3257
Atmospheres to sea water..........	33.096							

Specific gravities used in this table are: fresh water, 1.000; sea water, 1.025; mercury 13.5956.

A head of 1 ft. of water produces a pressure of 0.433 lb. per square inch. A head of 1 m. produces a pressure of 0.1 kg. per square centimeter.

For rough calculations, the weight of fresh water is frequently taken as 62.5 lb. per cubic foot; and one atmosphere equivalent to 34 ft. of fresh water, 33 ft. of sea water, or 30 in. of mercury.

THE FLOW OF WATER

The laws of hydraulics are essentially similar to the fundamental laws of mechanics. The basic principles governing the flow of water, neglecting the disturbing or modifying influences of friction and initial pressure, are founded upon the laws of falling bodies.

In 1643 Torricelli enunciated the theorem that "the velocity of a fluid passing through an orifice in the side of a reservoir is the same as that which is acquired by a body falling freely in vacuo from a vertical height measured from the surface of the fluid in the reservoir to the center of the orifice."[1]

In 1738 Daniel Bernoulli, the eminent mathematician of Basle, Switzerland, propounded the important hydraulic law of the conservation of energy in fluids, which may be stated thus: "At every section of a continuous and steady stream of frictionless fluid, the total energy is constant; whatever energy is lost as pressure is gained as velocity. Therefore, in terms of head: Total energy = velocity head + pressure head + head due to position = constant."[2]

Laws of Falling Bodies.—Neglecting the influence of friction, the laws of falling bodies are as follows:

$$v = gt = 32.16t \tag{1}$$

or, in words, the velocity of a falling body in a vacuum at any moment is equal to the time of the fall multiplied by the acceleration of gravity.

$$h = \frac{v}{2}t = \frac{1}{2}gt^2 = \frac{32.16}{2}t^2 = 16.08t^2 \tag{2}$$

where h = fall or vertical distance traveled in feet. The distance traversed, or the fall in feet, is equal to one-half of the product of the acceleration of gravity and the square of the time, in seconds, elapsing in the fall;

$$h = \frac{v^2}{2g} = \frac{v^2}{64.32} = 0.01555v^2 \tag{3}$$

or the distance traversed, or the fall in feet, or the velocity head, is equal to the square of the velocity divided by two times the acceleration of gravity;

$$v = \sqrt{2gh} = 8.02\sqrt{h} \tag{4}$$

[1] HUGHES and SAFFORD, "Hydraulics," 8.

[2] *Idem.*, 81.

or the velocity is equal to the square root of two times the fall in feet multiplied by the acceleration of gravity. The velocity then varies as the time and as the square root of the head, and the head varies as the square of the time and the square of the velocity.

If there be an initial velocity, V, in feet per second,

Equation (1) becomes

$$v = V + gt \qquad (5)$$

Equation (2) becomes

$$h = \tfrac{1}{2}gt^2 + Vt \qquad (6)$$

Equation (4) becomes

$$v = \sqrt{2gh} + V \qquad (7)$$

Flow of Water in Pipes and Conduits.—Water seeks its own level, the level or surface being approximately perpendicular to the direction of the force of gravity. Conversely, if its surface be not level, it will flow from the higher level to the lower. This is but another way of saying that difference in pressure, or in level or "head," as it is called technically, is necessary to make water flow—a fact sometimes overlooked.

If, then, there be available a certain difference in level—called "fall" if measured from the upper point to the lower, or "head" if measured from the lower to the upper—between two points along a pipe, conduit, or channel carrying water or any other liquid, flow will be induced at a rate dependent, first, upon the fall as compared with the length traversed; second, upon the cross-section of the pipe, conduit, or channel; third, upon the character of its interior surface; fourth, upon the condition of flow with reference to the pipe, *i.e.*, whether the pipe is under pressure or not, whether it is flowing full or partly full, and whether it is flowing uniformly, steadily, variably, or intermittently on account of constant or variable cross-section, or other cause; fifth, upon the presence or absence of curves or other partial obstructions, and upon the movement of air in partly filled pipes, or effect of wind in the case of open channels; and, sixth, upon the character, specific gravity, and viscosity of the liquid.

Let us examine briefly the hydraulic conditions of flow, first, in pipes flowing full or under pressure, *i.e.*, in pipes in which the pressure is outward, as in water pipes, and, second, in pipes or conduits flowing barely full or partly full in which there is a free surface of the liquid, as is the case with sewers.

Bernoulli's theorem, that the total energy in a steady stream of frictionless fluid is a constant and is equal to the elevation plus the velocity head plus the pressure head, may be expressed by the following formula:

$$H = h_e + h_v + h_p = h_e + \frac{v^2}{2g} + \frac{p}{w}$$

Practically, the conditions of the perfect fluid do not exist. Other elements enter the problem—the frictional resistance of the pipe, channel, or conduit to the flowing fluid, and other resistances to flow. These factors are covered in Bernoulli's theorem by the addition of other terms in the equation just given. As applied to two different points, A and B, between which there are no losses except that due to friction,

$H = h_e + h_v + h_p = h'_e + h' + h'_p + h_f$ using the same nomenclature and h_f being the loss in head due to the frictional resistance of the surface traversed by the fluid in passing from point A to point B

Resistances to Flow.—The more important elements of loss of head in pipes are the frictional loss due to the interior surface of the pipe, the loss on entrance into the pipe (called the "entry head"), and losses due to sudden enlargement, sudden contraction, changes in direction, partial obstructions, entrance of branches, etc. When the velocities are low, as is usually the case in sewers, losses other than that due to friction[1] are generally so small that they may be neglected, but in some cases, particularly when high velocities are involved, the velocity head and other losses may become of major importance. In long lines, the frictional loss is generally so much greater than the sum of all the others that it obscures the latter, and computations can frequently be made as though no other losses existed.

Hydraulic Grade Line.—The hydraulic grade line is a line connecting the points to which water would rise at various places along any pipe or conduit, were piezometer tubes, or vertical pipes open to the atmosphere, inserted in the liquid. It is a measure of the pressure head available at these points. The hydraulic grade line will, of course, be influenced not only by the frictional resistance due to the rugosity of the surface, but also by anything influencing the velocity head. In the case of a canal or open channel, in contradistinction to a pipe under pressure, the hydraulic grade line corresponds with the profile of its water surface.

Steady and Uniform Flow.—Steady flow exists in a conduit or stream when equal quantities of water pass the same point in like intervals of time, or, in other words, when the discharge is constant for successive intervals of time.

Uniform flow exists when the cross-section and the mean velocity of the flowing stream are the same at every point. Uniform flow is a steady flow in which the cross-sections of the stream are all alike, and its surface is parallel to its bed.

The difference in these two conditions of flow must be clearly borne in mind on account of its bearing upon loss of head due to various causes. It is illustrated by the comparison of flow through a pipe of

[1] Including that due to deposits or accumulations in the sewers

uniform diameter throughout its length and through a Venturi meter the ends of which are of similar diameter. While both may be discharging the same quantity of water, the flow in the former is uniform, in the latter, steady but nonuniform, due to its varying cross-section.

With few exceptions, the ordinary formulas relating to flow of water deal only with conditions of steady and uniform flow. Such conditions rarely exist in sewerage work, however, and it is necessary to give consideration to the effects of steady nonuniform flow, and also of unsteady flow. The former conditions always exist when there is a change in velocity resulting from a change in grade or of cross-section or where there is a partial obstruction in the conduit, and its effects may be experienced for a considerable distance; the latter conditions result from accretions to the quantity flowing, which are frequent in the case of most sewers.

In cases of unsteady or nonuniform flow, the only practicable method of computation is to assume the conduit divided into sections short enough so that the flow in each section may be assumed as steady and uniform without introducing serious error, and by successive approximations to obtain results close enough to meet the requirements of the problem. Some special cases of steady nonuniform flow frequently encountered, and sometimes of considerable importance, are discussed in a subsequent section of this chapter.

Critical Velocity.—Hughes and Safford state:[1]

Turbulent eddying motion exists in nearly all cases in practical hydraulic problems, and the resistance to flow varies in proportion to some power of the mean velocity between 1.7 and 2.0 or more. Certain investigations, however, have shown that at very low velocities the motion of the water is in parallel stream lines, that is, without the disturbance due to eddying motion; and the resistance to flow varies nearly directly as the mean velocity of flow. The velocity at which turbulent eddying motion begins or ceases is called the critical velocity.

Reynolds[2] made experiments to determine the point of critical velocity, and found that there were two critical values for any pipe or tube; "one at which steady motion changed into eddies, the other at which eddies changed into steady motion." The former change was found to occur at velocities considerably higher than the latter; and the two critical points are, therefore, called "the higher critical velocity" and "the lower critical velocity."

For the higher critical velocity,

$$v_c = \frac{1}{43.79}\frac{P}{D}\text{(meters per second)}$$

or

$$v_c = 0.2458\frac{P}{D}\text{ (feet per second)}$$

[1] "Hydraulics," First Edition, 90–92.

[2] Reynolds, Osborne, *Phil. Trans. Roy. Soc.*, 1883, pp. 935 et seq.

where

D = the diameter of the pipe

$P = (1 + 0.0336T + 0.000221T^2)^{-1}$ is the temperature correction

T = temperature of the water, degrees Centigrade

For the lower critical velocity,

$$v_c = \frac{1}{278}\frac{P}{D} \text{ (meters); or } v_c = 0.0387\frac{P}{D} \text{ (feet)}$$

Experiments by Barnes and Coker[1] show values for the higher critical velocity fully double those of Reynolds, and for the lower critical velocity as little as half as much as Reynolds'.

All these experiments showed that disturbances in the supply tank, or jarring of the pipes, made a marked change in the point of critical velocity. For practical conditions the point of critical velocity cannot be very precisely determined; and except for small pipes is usually too low to be considered.

Williams, Hubbell, and Fenkell[2] say:

. . . The experiments of Poiseuille, Hagen, Jacobson, and Hazen show that when water flows through capillary tubes or fine sands where it is prevented from taking up internal motions, because the area of the cross-section of the stream is almost molecular, that H_f varies very nearly as the first power of v. All reliable experiments on record show that as the diameter decreases the exponent of v, in $H_f = mv^n$, decreases, as has been shown for the lines investigated in this paper: 30 in., $H_f = mv^2$; 16 in., $H_f = mv^{1.86}$; 12 in., $H_f = mv^{1.78}$; and 2-in. brass, $H_f = mv^{1.76}$, from a possible limit of v^2. In other words, the more the chance for internal resistance, the higher the exponent of v. To the writers, then, the variation of the exponent of v is an index of the character of the flow, and when that becomes greater than unity, straight-line flow is over, or, the critical velocity of Professor Reynolds is past. If, then, these internal motions are capable of so increasing the rate of loss of head, it is evident that in them the controlling conditions of the laws of flow are to be looked for, rather than in the surface resistances. But, beyond this first critical velocity, there appear to be others where peculiar phenomena appear.

The resistance to flow for velocities under the critical velocity (the points at which eddying begins and ends) for capillary tubes and small pipes may be approximately computed by the following formula suggested by Allen Hazen.[3]

$$v = cSd^2\left(\frac{t + 10}{60}\right)$$

t = the temperature of the water, degrees Fahrenheit

c = a factor; from Saph and Schoder's experiments on brass pipes Hazen determined c to be from 462 to 584; Williams and Hazen use a value of 475 in their "Hydraulic Tables."

[1] *Proc. Roy. Soc.*, **74**, 341–356.

[2] Williams, Hubbell, and Fenkell, "Flow of Water in Pipes," *Trans. Am. Soc. C. E.*, 1902; **47**, 367.

[3] *Trans. Am. Soc. C. E.*, 1903; **51**, 316.

FRICTION AND FLOW OF WATER IN CONDUITS

Equation of Continuity.—The discharge of any conduit is given by the expression $Q = Av$

If the flow is steady in any given conduit there follows the equation of continuity

$$A_1v_1 = A_2v_2, \text{ etc.}$$

The term "velocity," when employed without qualification, is used throughout this discussion to signify the mean or average velocity in the entire cross-section. It is clear that as frictional resistance exists between the water and the walls of the pipe or conduit, the velocity of flow at these walls must be less than that in the center of the stream. The variation in the velocity at different points in the cross-section of a pipe discharging under pressure is shown in a general way in Fig. 12, and in a conduit flowing partly full in Fig. 13.

When there are no losses other than those due to friction, mean velocity is dependent upon, first, the available head or fall, and second, the resistance to the flowing stream.

The resistance in its turn varies with the length, wetted perimeter, and cross-section of the pipe, conduit, or channel; the rugosity, or roughness, of its interior surface; the temperature and hence viscosity of the fluid; and the condition of flow, uniform, steady, or variable. The resistance was shown by Dubuat to be independent of the water pressure, thus establishing the essential difference between the frictional resistance of a fluid and a solid as compared with the frictional resistance of two solids—the latter of which is dependent upon the weight or pressure of one solid upon the other.

Development of Formulas for Flow in Pipes and Channels.—All of the so-called formulas for flow are really expressions for the relation between the fall or slope of the hydraulic grade line and the velocity of flow in the conduit. From the velocity the discharge can readily be found by multiplying it by the area of the cross-section.

Ganguillet and Kutter[1] state:

The first attempt to discover the law by which the velocity of water depends upon the fall and the cross-section of the channel was, according to Hagen, made by Brahms (1753), who observed that the acceleration which we should expect in accordance with the law of gravity does not take place in streams, but that the water in them acquires a constant velocity. He points to the friction of the water against the wet perimeter as the force which opposes the acceleration, and assumes that its resistance is proportional to the mean radius R, *i.e.*, to the area of cross-section divided by the wet perimeter.

[1] "Flow of Water in Rivers and Other Channels," translated by Hering and Trautwine (1892).

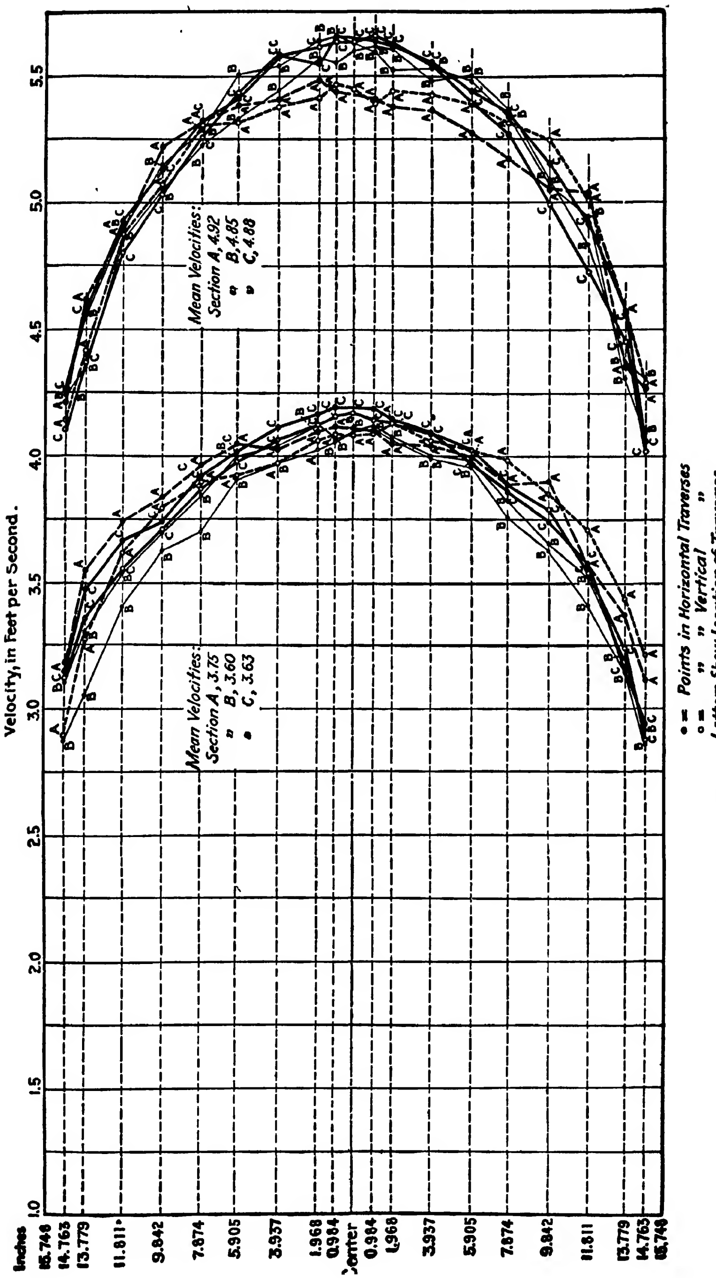


FIG. 12.—Distribution of velocities in 32-inch pipe. (*Bazin.*)
From Trans. Am. Soc. C. E., April, 1902; 47, 240.

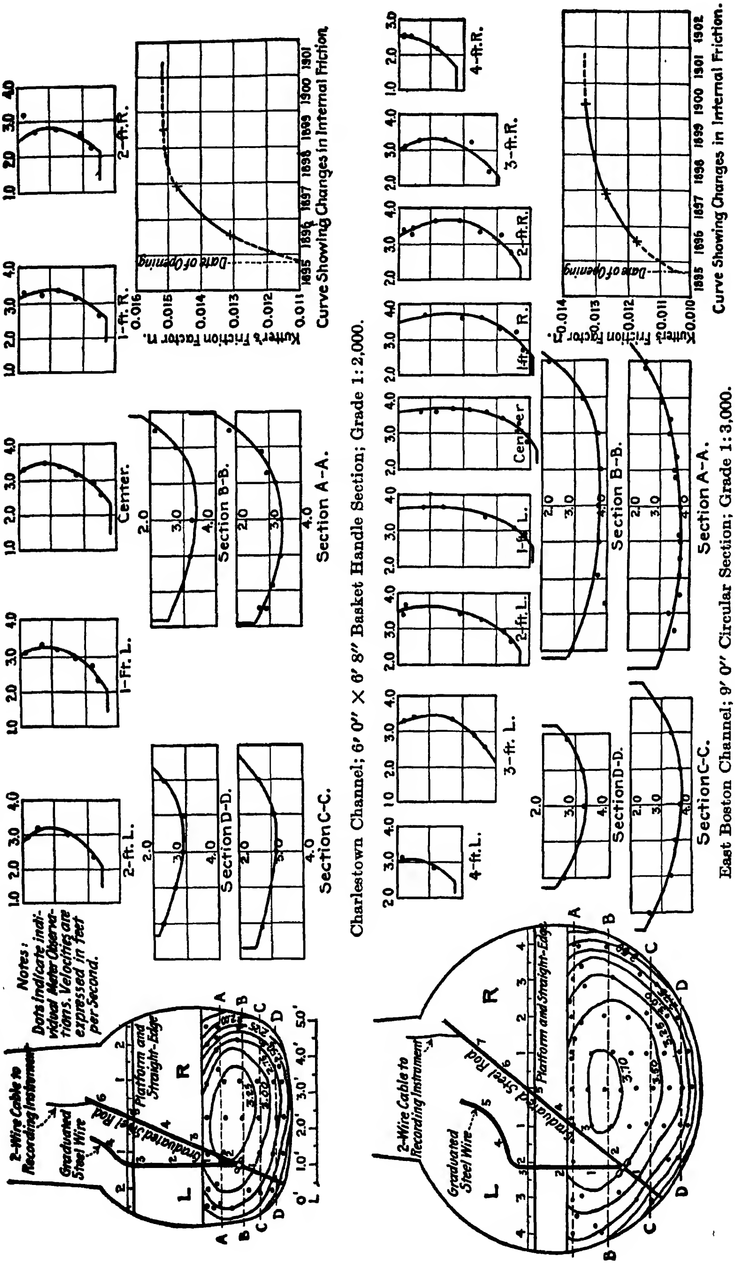


FIG. 13.—Velocity curves in two Metropolitan sewers. (*Boston.*)

Brahms and Chezy (1775) are to be regarded as the authors of the well-known formula

$$v = C\sqrt{\frac{A}{P} \times \frac{h}{L}} = C\sqrt{RS}$$

The principle established by Michelotti and Bossut that the laws governing the flow of water must be established experimentally, led Dubuat (1779) to investigate the flow of the Canal du Jard and the River Haine in France, and of experimental channels. He concluded that the force producing flow was the fall or slope of the water surface of the flowing stream, and that the resistance must be equal to this accelerating force, under conditions of uniform flow.

De Prony concluded:[1]

The particles of water in a vertical line in the cross-section of a stream move with different velocities, which diminish from the surface to the bottom.

The surface, bottom, and mean velocities stand in a certain relation to each other, which Dubuat, strange to say, finds to be independent of the size and form of the cross-section.

A layer of water adheres to the walls of the pipe or channel, and is therefore to be regarded as the wall proper which surrounds the flowing mass. According to Dubuat's experiments the adhesive attraction of the walls seems to cease at this layer, so that differences in the material of the walls produce no perceptible change in the resistance.

The particles of water attract each other mutually, and are themselves attracted by the walls of the channel. These attractions (resistances) may, in general, be expressed by means of two different values, which, however, are supposed to be the same nature and comparable with each other.

The idea that a film of water adhered to the walls of the channel, and that the friction was that of water upon water, and therefore independent of the character of the walls, persisted for a considerable period, but eventually it was noted by Darcy that the pipes having the smoothest interior surface delivered the greatest quantity of water and thus indicated the least frictional resistance to the flow. Believing that similar conditions must prevail in flowing streams, he began a series of experiments, which were continued after his death by his assistant, the famous hydraulic engineer, H. Bazin. These experiments demonstrated that the coefficient C in the Chezy formula was not a constant, as had previously been supposed, but varied with the character of the surface with which the water was in contact.

The various formulas for flow of water in pipes and channels are essentially empirical, and all of them apply to steady uniform flow, and consider only the losses due to friction.

In general, sewers are designed with the expectation that they will flow full only at times of maximum flow. The ordinary condition of

[1] GANGUILLET and KUTTER, "Flow of Water in Rivers and Other Channels," 4, 5.

flow is, therefore, that of an open channel, in which there is a free water surface in contact with air. When flowing full, they are usually under no material pressure, except in the case of force mains and inverted siphons.

The Chezy formula was intended to be applicable either to open channels or to pipes under pressure, and is perhaps as satisfactory for one case as for the other. The other formulas discussed herein were derived and originally applied either for open channels, or for pressure pipes; but their use has since been extended, and they may all be used with a good degree of satisfaction for open or closed channels such as are ordinarily considered in sewerage work.

The Chezy Formula.—This is

$$v = C\sqrt{RS}$$

where the coefficient C varies inversely with the smoothness of the wall of the channel, directly with R, and inversely with S in large streams but directly with it in small streams. C varies more rapidly with small values of R than with large.

The formula has long been the one most familiar to engineers, and as substantially all of the later results of experiments have been applied to it, as well as to some other formulas, the limits of its applicability have been better established than have those of any other formula for the flow of water in pipes, conduits, canals, and rivers.

The determination of the coefficient C under different conditions has received much study from hydraulicians. Indeed, several of the most widely used formulas are, in effect, the Chezy formula with additional terms for determining more readily the value of C.

The Chezy formula for the case of circular pipes flowing full may also be written in another form, which is attributed to Weisbach (see Coxe's translation of Weisbach's "Mechanics," p. 866).

$$h_f = f\frac{L}{D}\frac{v^2}{2g}$$

in which f = the coefficient of friction, which decreases with increase in pipe diameter and slightly with velocity of flow.

These two forms of the Chezy formula have been arranged by Hughes and Safford ("Hydraulics," 1911, p. 285), as applied to the flow of water in pipes, in the following manner:

$$v = C(RS)^{1/2}; \text{ or } h_f = \frac{fLv^2}{D2g}$$

For uniform steady flow in circular pipes:

the mean hydraulic radius, $R = \dfrac{D}{4}$

the slope of the hydraulic grade line, $S = \dfrac{h_f}{L}$

the area of the stream, $A = \dfrac{\pi D^2}{4}$ Then:

the friction head, $h_f = \frac{4v^2L}{C^2D}$; or $h_f = f\frac{Lv^2}{D2g}$

the mean velocity of flow in feet per second,

$$v = \frac{C}{2}\left(\frac{h_fD}{L}\right)^{1/2}; \text{ or } v = 8.02\left(\frac{h_fD}{fL}\right)^{1/2}$$

the discharge in cubic feet per second, $Q = Av = \frac{\pi D^2v}{4}$

$$Q = 0.3927C\left(\frac{h_fD^5}{L}\right)^{1/2}; \text{ or } Q = 6.3\left(\frac{h_fD^5}{fL}\right)^{1/2}$$

the diameter in feet required to deliver a given discharge,

$$D = 1.453\left(\frac{LQ^2}{h_fC^2}\right)^{1/5} \text{ or } D = 0.479\left(\frac{fLQ^2}{h_f}\right)^{1/5}$$

Comparison of coefficients C and f,

$$C = \left(\frac{8g}{f}\right)^{1/2} = \frac{16.04}{(f)^{1/2}}; \text{ and } f = \frac{8g}{C^2} = \frac{257.28}{C^2}$$

The Kutter Formula.—Among those who have given study to the correct determination of the coefficient C to be used in the Chezy formula for the flow of water in pipes, conduits, and channels, were the Swiss engineers Ganguillet and Kutter, of Berne. Their results were first published in a series of articles in the German technical press. They were translated into English by L. D'A. Jackson (London, 1876), and again by Rudolph Hering and J. C. Trautwine, Jr., in 1892, who presented them with additions in a volume entitled, "A General Formula for the Uniform Flow of Water in Rivers and Other Channels, by E. Ganguillet and W. R. Kutter, Translated from the German, With Numerous Additions including Tables and Diagrams, and the Elements of over 1,200 Gagings of Rivers, Small Channels, and Pipes."

In its general form, the formula, which is known as Kutter's, is

$$v = \left(\frac{a + \frac{l}{n} + \frac{m}{S}}{1 + \left(a + \frac{m}{S}\right)\frac{n}{\sqrt{R}}}\right)\sqrt{RS}$$

The values a, l, and m are constant and n varies with the degree of roughness. Substituting the numerical values found for the constants, we have in English measure.

$$v = \left(\frac{41.66 + \frac{1.811}{n} + \frac{0.00281}{S}}{1 + \left(41.66 + \frac{0.00281}{S}\right)\frac{n}{\sqrt{R}}}\right)\sqrt{RS}$$

Values of n in Kutter's Formula.—For coefficients of roughness, n, with their reciprocals, etc., Ganguillet and Kutter suggested the values in Table 7.

TABLE 7.—VALUES OF n IN KUTTER FORMULA RECOMMENDED BY GANGUILLET AND KUTTER

	n	$\frac{1}{n}$
1. Channels lined with carefully planed boards or with smooth cement	0.010	100.00
2. Channels lined with common boards	0.012	83.33
3. Channels lined with ashlar or with neatly jointed brickwork	0.013	76.91
4. Channels in rubble masonry	0.017	58.82
5. Channels in earth, brooks, and rivers	0.025	40.00
6. Streams with detritus or aquatic plants	0.030	33.33

These values are not sufficient to enable a proper choice to be made, and are not altogether consistent with the results of more recent experiments. The best modern list of values of n is that given by Robert E. Horton,[1] which is reproduced as Table 8, as printed in King's "Handbook of Hydraulics."

It will be noted that the table includes values of n for flow in pipes of cast iron and other materials. The formula was not devised for pipes under pressure, and is not recommended for such use. It has been found, however, that it can be used for such cases with a fair degree of satisfaction, particularly when the pressure is slight. Its use should preferably be limited to open channels, including closed conduits or pipes not flowing full or under pressure.

Most of the experimental determinations of the value of n have been made on conduits for carrying clean water, and upon irrigation and drainage canals and ditches. The results of a very large proportion of such experiments may be found in *Bulls.* 194 and 852 of the U. S. Department of Agriculture, by Fred C. Scobey. Attention should also be called to *Bull.* 854 on "The Flow of Water in Drain Tile," by D. L. Yarnell and S. M. Woodward, which contains the results of a very comprehensive series of experiments on drain tile, 4 in. to 12 in. in diameter, with slopes varying from 0.0005 to 0.015, and with depths of flow varying from about one-fourth of the diameter to full. These were laboratory experiments carried out with great care, but the results with flow at part depth varied considerably. They showed that, in general, the value of n was greater when the drain was partly filled than when entirely filled, the excess at 0.25 depth being roughly from 10 to 50 per cent; but there were a few cases in which lower values of n for less depths of flow were found.

[1] *Eng. News*, 1916; **75**, 373.

TABLE 8.—R. E. HORTON'S VALUES OF n; TO BE USED WITH KUTTER'S FORMULA

Surface	Best	Good	Fair	Bad
Uncoated cast-iron pipe	0.012	0.013	0.014	0.015
Coated cast-iron pipe	0.011	0.012[1]	0.013[1]	
Commercial wrought-iron pipe, black	0.012	0.013	0.014	0.015
Commercial wrought-iron pipe, galvanized	0.013	0.014	0.015	0.017
Smooth brass and glass pipe	0.009	0.010	0.011	0.013
Smooth lockbar and welded "OD" pipe	0.010	0.011[1]	0.013[1]	
Riveted and spiral steel pipe	0.013	0.015[1]	0.017[1]	
Vitrified sewer pipe	0.010 / 0.011	0.013[1]	0.015	0.017
Common clay drainage tile	0.011	0.012[1]	0.014[1]	0.017
Glazed brickwork	0.011	0.012	0.013[1]	0.015
Brick in cement mortar; brick sewers	0.012	0.013	0.015[1]	0.017
Neat cement surfaces	0 010	0.011	0.012	0.013
Cement-mortar surfaces	0.011	0.012	0.013[1]	0.015
Concrete pipe	0.012	0.013	0.015[1]	0.016
Wood-stave pipe	0.010	0.011	0.012	0.013
Plank flumes:				
Planed	0.010	0.012[1]	0.013	0.014
Unplaned	0.011	0.013[1]	0.014	0.015
With battens	0.012	0.015[1]	0.016	
Concrete-lined channels	0.012	0.014[1]	0.016[1]	0.018
Cement-rubble surface	0.017	0.020	0.025	0.030
Dry rubble surface	0.025	0.030	0.033	0.035
Dressed ashlar surface	0.013	0.014	0.015	0.017
Semicircular metal flumes, smooth	0.011	0.012	0.013	0.015
Semicircular metal flumes, corrugated	0.0225	0.025	0.0275	0.030
Canals and ditches:				
Earth, straight and uniform	0.017	0.020	0.0225[1]	0.025
Rock cuts, smooth and uniform	0.025	0.030	0.033[1]	0.035
Rock cuts, jagged and irregular	0.035	0.040	0.045	
Winding sluggish canals	0.0225	0.025[1]	0.0275	0.030
Dredged earth channels	0.025	0.0275[1]	0.030	0.033
Canals with rough stony beds, weeds on earth banks	0.025	0.030	0.035[1]	0.040
Earth bottom, rubble sides	0.028	0.030[1]	0.033[1]	0.035
Natural stream channels:				
1. Clean, straight bank, full stage, no rifts or deep pools	0.025	0.0275	0.030	0.033
2. Same as (1), but some weeds and stones	0.030	0.033	0.035	0.040
3. Winding, some pools and shoals, clean	0.033	0.035	0.040	0.045
4. Same as (3), lower stages, more ineffective slope and sections	0.040	0.045	0.050	0.055
5. Same as (3), some weeds and stones	0.035	0.040	0.045	0.050
6. Same as (4), stony sections	0.045	0.050	0.055	0.060
7. Sluggish river reaches, rather weedy or with very deep pools	0.050	0.060	0.070	0.080
8. Very weedy reaches	0.075	0.100	0.125	0.150

[1] Values commonly used in designing (according to Horton; the authors' recommendations will be found on p. 90).

All of the determinations of the value of n which have been made on sewers in use, which have come to the knowledge of the authors, are summarized in the following pages.

Theodore Horton (1901), in an admirable article upon "Flow in the Sewers of the North Metropolitan Sewerage System of Massachusetts,"[1] gives an account of gagings made in the Metropolitan sewers with a current meter.

The points selected for carrying out these observations were at manholes located some distance below the pumping stations, where the flow was free from any disturbing influence of the pumps. The points were about 800 ft. below the stations, in each case, and were far removed from any changes in alignment, cross-section, or grade of the sewer. Below the East Boston pumping station the cross-section of the sewer is a 9-ft. circle of 12-in. brickwork, cement-washed, with a hydraulic gradient of 1:3,000. Only one small local connection enters this stretch of sewer. No changes in grade occur within a distance of 7,000 ft. below the pumping station. At a point 2,000 ft. below the pumping station, there is a change from a circular section to a horseshoe section of the same equivalent area. This section continues for a distance of 2,000 ft. and then returns to a circular section. Below the Charlestown pumping station, the cross-section of the sewer is 6 ft. by 6 ft. 8 in., baskethandle, of 8-in. brickwork, cement-washed, with a hydraulic gradient of 1:2,000. The cross-section and grade are uniform for a distance of about 5,000 ft. below the pumping station, and no local connections enter the sewer within this distance.

The results of the test are shown in Table 9 and Fig. 13.

TABLE 9.—VALUES OF n IN KUTTER'S FORMULA, DETERMINED FROM GAGINGS OF A CEMENT-WASHED BRICK TRUNK SEWER (THEODORE HORTON)

Series of July, 1896—Charlestown Pumping Station

No.	Depth	Q in cu. ft. per sec.	Mean velocity	Hydraulic radius	C	n
I	1.02	8.60	1.99	0.688	107	0.0129
II	1.44	16.59	2.46	0.958	112	0.0131
III	1.91	26.81	2.82	1.187	115	0.0132
IV	2.40	38.82	3.13	1.387	118	0.0133
V	2.89	52.90	3.44	1.539	124	0.0130

[1] *Trans. Am. Soc. C. E.*, 1901; **46**, 78.

TABLE 9.—VALUES OF n IN KUTTER'S FORMUAL, DETERMINED FROM GAGINGS OF A CEMENT-WASHED BRICK TRUNK SEWER (THEODORE HORTON).—(*Continued*)

No.	Depth	Q in cu. ft. per sec.	Mean velocity	Hydraulic radius	C	n
Series of July, 1896—East Boston Pumping Station						
I	1.02	6.10	1.58	0.619	110	0.0122
II	1.52	15.67	2.21	0.928	126	0.0117
III	2.04	29.40	2.70	1.208	134	0.0116
IV	2.45	42.18	3.03	1.408	139	0.0115
V	3.16	69.50	3.48	1.830	141	0.0117
VI	3.75	94.60	3.73	1.999	145	0.0113
VII	4.62	138.00	4.18	2.309	150	0.0115
Series of December, 1897—Charlestown Pumping Station						
I	2.91	45.67	2.97	1.540	107	0.0149
II	3.29	56.14	3.16	1.650	111	0.0147
Series of November, 1897—East Boston Pumping Station						
I	2.15	30.13	2.55	1.280	123	0.0126
II	2.74	47.75	2.90	1.560	127	0.0127
III	3.19	62.05	3.06	1.762	126	0.0129
IV	3.20	64.82	3.18	1.771	131	0.0126
Series of June, 1900—Charlestown Pumping Station						
I	2.29	30.82	2.66	1.342	102	0.0151
II	2.78	41.39	2.86	1.508	104	0.0152
III	3.26	52.96	3.04	1.645	106	0.0152
Series of April, 1900—East Boston Pumping Station						
I	1.99	24.96	2.38	1.120	119	0.0130
II	2.83	48.26	2.82	1.606	121	0.0132
III	3.64	76.78	3.16	1.952	124	0.0133
IV	4.18	95.84	3.30	2.130	124	0.0136

Horton concluded, among other things, that the greatest change in internal surface of the sewers took place soon after the channels were put into operation, the initial coefficient of friction n for use in Kutter's formula being between 0.010 and 0.011, the Charlestown channel giving slightly the higher value. In comparing these changes in the values of n with the actual condition of the channels, it should be kept in mind that:

The East Boston channel is of 3 ft. greater diameter than the Charlestown channel, that the invert of the East Boston channel is approximately 3 ft. above mean low water, while the Charlestown channel is 4 ft. below mean low water, and that the East Boston channel receives relatively less storm water than the Charlestown channel, and is, consequently, subject

to less fluctuation of water surface. The importance of this last influence is evident from the fact that the deposit of both grease and organic growth appeared in greater abundance on the sides of the channels, and was greatest near the line of average flow. On the bottom of the channel there was practically no deposit, resulting, no doubt, from the scouring action of sand and other heavy particles transported along the invert by the sewage. This last feature is by no means novel, and has frequently been observed, though to a less extent, in water-supply conduits.

The effect of the density of the sewage upon the carrying capacity of these channels appears to be slight, in view of the fact that the observations were made under all the varying conditions of storm and dry-weather flow. The possible effect of cleaning or scraping, however, might be much greater, but, at this date, no cleaning of any sort has taken place in these channels.

The Bureau of Surveys of Philadelphia, Pa. (1909) had a series of observations made, of the values of the coefficient of roughness, n, of certain of the large sewers in that city, with the following result:

	n
Old sewers, brick bottom, not clean	0.017
Old sewers, stone-block bottom, clean	0.017
New sewers, stone-block bottom, clean	0.016
New sewers, brick bottom, clean	0.015
Concrete or brick sewer, vitrified-shale brick invert, clean	0.012 to 0.013
Concrete sewers, granolithic-finished bottom	0.011
Open channel box, planed planks	0.011
Old sewers, bad or dirty bottoms	0.017 to 0.020

T. Chalkley Hatton reported the results of experiments on the flow of water in two 24-in. sewers built with 3-ft. lengths of pipe and with cement joints, at Carlisle, Pa. Experiments on a section 4,660 ft. long having a grade of 0.077 per cent, and having bends at five manholes, with depths of water of 5 and 12 in., gave $n = 0.0128$ and $n = 0.0112$, respectively. One experiment on another section 2,095 ft. long and having one bend at a manhole and a grade of 0.04 per cent, gave with a depth of 12 in. $n = 0.0111$, as computed by the authors from Hatton's data.

Alexander Potter reported that for vitrified pipe and small brick sewers the coefficient of roughness ranged from 0.013 to 0.0145, and the value of 0.014 represented average conditions of roughness and depth of flow found in practice, as shown by the results of observations made on the joint trunk sewer system in northeastern New Jersey, where the contributing flows from various municipalities are measured by 13 automatic gages keeping a continuous record of the depth of the discharge. Once a week the charts are taken out and new blanks substituted, and as a check on the readings of each chart the operator determines the actual depth of flow and, by means of floats, the velocity of

the sewage at that point. The average results of 50 to 60 observations made in 1906 to 1909, inclusive, on sewers built in 1903, are given in Table 10.

TABLE 10.—MEASURED AND COMPUTED VELOCITIES AND THEIR PERCENTAGE RATIO (POTTER)

m.v., measured velocity, feet per second; *c.v.*, computed velocity; *p.r.*, percentage ratio

Ratio of depth of flow to diameter		0.20	0.30	0.40	0.50	0.60	0.70
Gage No. 60	*m.v.*	....	....	3.18	3.45	3.65	3.84
42-in. brick sewer	*c.v.*	....	....	3.08	3.49	3.77	3.98
0.13 % grade	*p.r.*	....	....	103.2	98.9	96.8	96.5
Gage No. 53½	*m.v.*	....	2.36	2.70	2.92	3.20	3.36
20-in. pipe sewer	*c.v.*	....	2.20	2.60	3.05	3.30	3.48
0.28 % grade	*p.r.*	....	107.2	100.5	95.7	97.0	96.6
Gage No. 4½	*m.v.*	....	....	4.23	4.81	5.20	5.54
22-in. pipe sewer	*c.v.*	....	....	4.27	4.85	5.24	5.53
0.6 % grade	*p.r.*	....	....	99.1	99.2	99.2	100.2
Gage No. 35	*m.v.*	....	2.18	2.50	2.73	2.90	
24-in. pipe sewer	*c.v.*	....	1.96	2.40	2.72	2.94	
0.18 % grade	*p.r.*	....	111.3	104.2	100.4	98.7	
Gage No. 72	*m.v.*	1.78	2.05	2.58			
22-in. pipe sewer	*c.v.*	1 62	2.08	2.55			
0.22 % grade	*p.r.*	109.8	99.7	98.1			

Percentage ratio of 109 corresponds to $n = 0.013$.
Percentage ratio of 100 corresponds to $n = 0.014$
Percentage ratio of 92 corresponds to $n = 0.015$.

Vitrified pipe sewers in Cambridge, Mass., part of the Metropolitan Sewerage Works, tested by the authors in 1923, had the characteristics shown in the following table:

TABLE 11.—VALUES OF n IN 15- AND 18-IN. PIPE SEWERS AT CAMBRIDGE, MASS.

Diameter, inches	Length, feet	Proportional depth of flow	R, feet	Slope	Velocity, feet per second	Chesy C	Kutter n
15	2,459	0.475	0.302	0 000915	1.64	99	0.0121
18	1,836	0.545	0.396	0.000844	1.66	91	0.0135

Velocities were measured by observing the time of passage of eosine dye. Both sewers are on uniform grades, and the deflections in alignment are inconsiderable. There are no branches on either line. There are 19 intermediate manholes on the 15-in. sewer, and 9 on the 18-in. sewer, with benches at about the elevation of the center of the pipe;

consequently, they caused some slight disturbance of the flow in the 18-in. sewer, where the depth of flow was above the center, and none in the 15-in. sewer where the depth was below the center. Both sewers were in good condition, but neither had been cleaned for several months, and the 15-in. pipe was noticeably cleaner than the 18-in. There was a barely perceptible current of air in the direction of flow in both pipes.

S. M. Cotten[1] reported experiments on flow in the main outlet sewers of Phoenix, Arizona, consisting of vitrified clay pipes 24, 30, and 36 in. in diameter. Velocities were measured by timing the passage of a plunger float, which nearly filled the sewer. The principal data relating to the tests (including information obtained from Mr. Cotten, not given in the published paper) are as follows:

TABLE 12.—VALUES OF n IN 24-, 30-, AND 36-IN. PIPE SEWERS AT PHOENIX, ARIZ.

Diameter, inches	Length, feet	Proportional depth of flow	Velocity, feet per second	Kutter, n
24	1,000	0.455	2 74	0.0111
30	2,693	0.355	2 39	0.0117
36	1,173	0.450	2.06	0.0125

These sewers are straight and on uniform slopes. There are no branches or other causes of disturbance in flow, and conditions were good for minimum values of n.

H. D. Silliman[2] in a letter describing sewer gagings at Seattle, Wash., said:

> The running time . . . was calculated by using Kutter's constants, $n = 0.013$ for pipe sewers and $n = 0.015$ for brick sewers . . . In all the velocity measurements that I have made, I have not found an old sewer that had more than 90 per cent of its calculated velocity using the foregoing constants. Some ran as low as 75 per cent of the calculated velocity.

If 85 per cent of the computed velocity with $n = 0.013$ be taken as a fair representation of the average observed velocity in pipe sewers, this would agree with the result obtained (in the case of a 12-in. pipe) by using $n = 0.015$. That is to say, Mr. Silliman's tests indicate that the observed velocities were roughly in accord with $n = 0.015$ for pipe sewers.

S. T. Weller[3] describes gagings of flow in the four main outlet sewers of Denver, Colo., which were made by current meter at times when the

[1] *Eng. News Rec.*, 1922; **89**, 837.
[2] *Eng. News*, 1915; **74**, 1093.
[3] *Eng. News Rec.*, 1928; **100**, 557.

sewage was practically clear water. Three of these sewers are of brick, the fourth of vitrified pipe. The values found for n are as follows:

Sewer	Material	Size, in.	Value of n		
			High	Low	Average
Delgany St.	Brick	77	0.0134	0.0131	0.0133
South and West Side	Brick	70	0.1030	0.0102	0.0103
East Side	Brick	38	0.0136	0.0131	0.0133
Berkeley	Vit. pipe	21	0.0156	0.0152	0.0156

Keefer and Regester,[1] experimenting on flow in a 42-in. cast iron sewage force main at Baltimore, in use for 15 years, found values of n ranging from 0.0139 to 0.0144, based upon the nominal diameter, although the actual diameter was about 2 inches less owing to the accumulation of a layer of slime and grease. The values of C_h in the Hazen and Williams formula were from 102 to 104.

Brooklyn, N. Y.—Current meter measurements of dry weather flow in a combined sewer were made in order to determine the value of n to be used in gagings of storm water flow based upon records of water stage at two points. The sewer is of brick, with egg-shaped section 3.9 by 5.5 ft., and the average value of n for the portion of the invert covered by the dry weather flow was found to be 0.016.[2]

Effect of Variation in Assumed Value of n.—Ernest W. Schoder[3] called attention to the fact that the percentage error resulting from a wrong assumption as to the value of the coefficient of roughness n can readily be approximately determined for the Kutter formula in spite of the apparently complicated nature of its coefficients. Broadly speaking, the following relations hold:

1. The slope S varies as n^2, almost exactly for all values of the hydraulic radius R greater than 1 ft.

2. The velocity v varies inversely as n, exactly for R = about 2 ft., and approximately for other values.

Corresponding to these relations, we may state that a certain percentage of uncertainty in the value of n produces:

1. Double that percentage of uncertainty in the slope necessary for a fixed discharge.

2. The same percentage of uncertainty, but in opposite direction, in the velocity of discharge resulting from a fixed slope, if the slope is assumed to be greater than 0.0001.

[1] *Eng. News Rec.*, 1928; **100**, 360.

[2] *Public Works*, 1927; **58**, 430.

[3] *Eng. News*, 1912; **68**, 351.

As an illustration of the convenience of this knowledge, suppose that in designing a canal, it is uncertain what value in the range between 0.017 and 0.020 to choose for n. This is an uncertainty of about 8 per cent either way from the mean value and represents a probable occurrence in practice. We can state at once that the uncertainty in discharge as caused by ignorance concerning n will be about 8 per cent and in required slope, about 16 per cent.

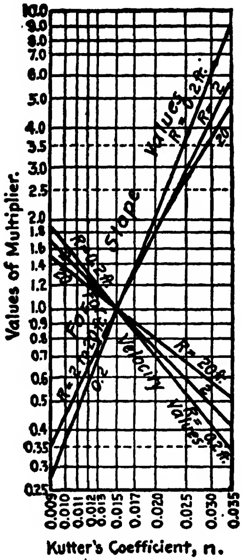


FIG. 14.—Relation between Kutter's n and corresponding slopes and velocities.

The diagram prepared by Schoder is given in Fig. 14; reference may also be made to diagrams 5 and 15 of Swan and Horton's "Hydraulic Diagrams," First Ed.

Figure 14 shows that, for $R = 2.0$, the velocity (or discharge) will be 1.08 times as much for $n = .013$ as for $n = .015$; and 1.34 times as much for $n = .010$ as for $n = .013$ (since $1.45 \div 1.08 = 1.34$). Other comparisons may be made in a similar way.

Table 13, which gives the mean percentage relation between the velocities or discharges for various values of n, will also be of service for approximate comparison. This table has been computed by averaging the percentage relations computed for slopes of 0.0001, 0.001, and 0.01, and hydraulic radii of 0.2, 0.6, 1.0, 2.0, and 4.0.

TABLE 13.—CONVERSION FACTORS FOR VARYING VALUES OF n

Revised value of n	Per cent of Q or v for original value of n =			
	0.015	0.013	0.012	0.011
0.0090	179	152	140	126
0.0100	159	135	124	112
0.0110	142	121	111	100
0.0120	128	109	100	90
0.0130	118	100	92	83
0.0140	108	92	84	76
0.0150	100	85	78	70
0.0160	93	79	73	65
0.0170	87	74	68	61
0.0180	81	69	63	57
0.0190	77	65	60	54
0.0200	72	61	56	50

As an example, a sewer was designed using $n = 0.013$. Later data indicated that a value of $n = 0.012$ was more suitable. In column 3, opposite the value 0.012 in column 1, we find that the revised discharge is 109 per cent of the discharge as originally computed.

Limitations of Kutter's Formula.—Being essentially an empirical formula, based upon actual gagings, it is of importance to remember the limits within which observations have been made and further to remember that while velocity varies approximately as the square root of the head for velocities corresponding to the ordinary conditions of flow, it varies more nearly directly as the head for extremely low velocities. Within the ordinary velocity limits of from 1 to 6 ft. per second, the formula finds its best application. It is fairly reliable up to 10 ft. per second velocity. For special cases, which may be outside of the range of the formula such as 20 ft. per second or higher velocity, the engineer should refer to the original data, published in Hering and Trautwine's translation of Ganguillet and Kutter's work, and that of other writers upon hydraulics since that time.

Hughes and Safford[1] have summed up the application of this formula in an excellent manner as follows:

There is a wide range in the magnitude of the streams on which this formula is based (from hydraulic radii of 0.28 to 74.4 ft.); but a study of the data on which the formula is based, as given in the authors' book, has led to the following conclusions:

That, for hydraulic radii greater than 10 ft., or velocities higher than 10 ft. per second, or slopes flatter than 1 in 10,000, the formula should be used with great caution. For hydraulic radii greater than 20 ft., or velocities higher than 20 ft. per second, but little confidence can be placed in results.

That, considering the variable accuracy of the data on which the formula is based, results should not be expected to be consistently accurate within less than about 5 per cent.

That, for any slope steeper than 0.001 the values of C computed for $S = 0.001$ may be used with errors less than the probable error in the ordinary use of Kutter's formula.

That between slopes of 0.001 and 0.0004 the maximum variation (at the extreme values of n and R) in C is about 4 per cent; for such values as fall within the range of ordinary practice, the maximum variation is but 2 per cent.

That between slopes of 0.0004 and 0.0002 the maximum variation is about 5 per cent, but for such values as fall within the range of ordinary practice the maximum is less than 3 per cent.

That for higher values of S the divergence in the values of C increases; but the occasions when slopes flatter than 0.0004 are to be considered in design are not common, and when they do occur they are usually for structures of such high character that they warrant special study and some basis in addition to a general empirical coefficient. And considering that a degree of

[1] "Hydraulics," First Edition, 343.

precision of 0.001 is rarely exceeded in leveling for ordinary construction work, and that in picking out the value of n, a variation of 0.001 for small values of n and R may change the value of C as much as 17 per cent, and for moderate values as much as 5 to 8 per cent, it should be obvious that hair-splitting calculations with the Kutter formula are a needless waste of time, producing merely mechanical accuracy instead of a high degree of precision.

Suggested Values of n for Sewer Design.—In view of the facts cited, the authors suggest the following as reasonable values for the coefficient of roughness n in Kutter's formula to be used in the design of sewer pipes, conduits, and channels, *to be maintained in reasonably good operating condition.*

	n
For pipe sewers 24 in. or less in size	0.015
For concrete sewers of large section and best work	0.012
For concrete sewers under good ordinary conditions of work	0.013
For brick sewers lined with vitrified or reasonably smooth hard-burned brick and laid with great care, with close joints	0.014
For brick sewers, under ordinary conditions	0.015
For brick sewers, rough work	0.017 to 0.020

The value $n = 0.013$ for vitrified pipe sewers has probably been used in design more than any other. While this is a reasonable value to represent the effect of friction in the pipes, the practical necessity of including in the value of n the effect of other influences, especially manholes, branches and changes of direction, makes it desirable to use $n = 0.015$ to ensure the desired capacity. The considerable losses which may occur at manholes make it particularly advantageous to place the bench at the elevation of the crown, rather than at the center of the sewer. The possible retarding effect of currents of air in sewers partly filled must also be borne in mind.

In the case of the larger sewers of concrete or brick, manholes do not often involve enlargements of cross-section, branches are likely to enter nearly tangentially, and other disturbing influences are smaller proportionally than in the case of pipe sewers, and an increase in the value of n to cover their effect is seldom necessary.

Solution of the Kutter Formula.—The complexity of the formula makes its arithmetical solution difficult, but tables and diagrams have been prepared by which results can easily be obtained. Tables giving the value of C corresponding to various values of S, R, and n, and covering a range beyond any likely to be experienced in ordinary sewerage work, may be found in King's "Handbook of Hydraulics" and most of the engineer's pocketbooks.

The Kutter formula is most readily used by means of diagrams. For many years these were collections of curves plotted on ordinary

cross-section paper. The advantages of logarithmic paper for plotting such formulas gradually became recognized, and in 1901 John H. Gregory prepared the diagrams shown in Figs. 15 to 20, which are part of a series of labor-saving charts devised by him at that time. The logarithmic method of plotting has been used by the authors in constructing Figs. 21, 22, and 23, in which the arrangement adopted results in a more open diagram than those of Professor Gregory.

These diagrams have been prepared especially for sewer design, and enable most computations of sewer flow to be readily solved. They are not suited for the general solution of Kutter's formula. For this, such a diagram as that shown in Fig. 24, prepared by Fred C. Scobey, Senior Irrigation Engineer of the U. S. Department of Agriculture,[1] and somewhat extended by him from one published in Bull. 852 of the Department, will be found advantageous; or arithmetical computation can be made with the aid of tables.

Dotted lines on the diagram show the method of use for one problem. It will be noted that the "guide lines," which are straight throughout the greater part of the chart, split into two curves for values of S less than about 0.0003; these curves have been drawn for $n = 0.012$ and $n = 0.030$ respectively, and interpolation can be made by eye for intermediate values of n.

The areas and hydraulic radii of circular and egg-shaped sections partly filled are given in Tables 14, 15, and 16. These data are shown graphically for these and other shapes of section, as well as the velocity and discharge at various depths for certain slopes and values of n, in the diagrams in Chap. III.

TABLE 14.—AREAS OF CIRCULAR SECTIONS PARTLY FILLED
Hughes and Safford's "Hydraulics"

Ratio of depth to diameter	Factors by which to multiply D^2 to obtain area									
	0.00	0.01	0.02	0.03	0.04	0.05	0.06	0.07	0.08	0.09
0.0	0.0000	0.0013	0.0037	0.0069	0.0105	0.0147	0.0192	0.0242	0.0294	0.0350
0.1	0.0409	0.0470	0.0534	0.0600	0.0668	0.0739	0.0811	0.0885	0.0961	0.1039
0.2	0.1118	0.1199	0.1281	0.1365	0.1449	0.1535	0.1623	0.1711	0.1800	0.1890
0.3	0.1982	0.2074	0.2167	0.2260	0.2355	0.2450	0.2545	0.2642	0.2739	0.2836
0.4	0.2934	0.3032	0.3130	0.3229	0.3328	0.3428	0.3527	0.3627	0.3727	0.3827
0.5	0.3927	0.4027	0.4127	0.4227	0.4327	0.4426	0.4526	0.4625	0.4724	0.4822
0.6	0.4920	0.5018	0.5115	0.5212	0.5308	0.5404	0.5499	0.5594	0.5687	0.5780
0.7	0.5872	0.5964	0.6054	0.6143	0.6231	0.6319	0.6405	0.6489	0.6573	0.6655
0.8	0.6736	0.6815	0.6893	0.6969	0.7043	0.7115	0.7186	0.7254	0.7320	0.7384
0.9	0.7445	0.7504	0.7560	0.7612	0.7662	0.7707	0.7749	0.7785	0.7816	0.7841
1.0	0.7854									

[1] For permission to reproduce this diagram the authors are indebted to the Division of Agricultural Engineering, U. S. Department of Agriculture and to Mr. Scobey. It was first printed in its present form in Creager and Justin's "Hydroelectric Handbook," 1927.

Table 15.—Hydraulic Mean Radii of Circular Sections Partly Filled

Hughes and Safford's "Hydraulics"

Ratio of depth to diameter	Factor by which to multiply D to obtain R									
	0.00	0.01	0.02	0.03	0.04	0.05	0.06	0.07	0.08	0.09
0.0	0.0000	0.0066	0.0132	0.0197	0.0262	0.0326	0.0389	0.0451	0.0513	0.0575
0.1	0.0635	0.0695	0.0755	0.0813	0.0871	0.0929	0.0986	0.1042	0.1097	0.1152
0.2	0.1206	0.1259	0.1312	0.1364	0.1416	0.1466	0.1516	0.1566	0.1614	0.1662
0.3	0.1709	0.1756	0.1802	0.1847	0.1891	0.1935	0.1978	0.2020	0.2062	0.2102
0.4	0.2142	0.2181	0.2220	0.2258	0.2295	0.2331	0.2366	0.2401	0.2435	0.2468
0.5	0.2500	0.2531	0.2562	0.2592	0.2621	0.2649	0.2676	0.2703	0.2728	0.2753
0.6	0.2776	0.2799	0.2821	0.2842	0.2862	0.2881	0.2900	0.2917	0.2933	0.2948
0.7	0.2962	0.2975	0.2987	0.2998	0.3008	0.3017	0.3024	0.3031	0.3036	0.3039
0.8	0.3042	0.3043	0.3043	0.3041	0.3038	0.3033	0.3026	0.3018	0.3007	0.2995
0.9	0.2980	0.2963	0.2944	0.2921	0.2895	0.2865	0.2829	0.2787	0.2735	0.2666
1.0	0.2500									

Table 16.—Elements of Egg-shaped Section for Various Depths of Flow

For section in which width = ⅔ of height, and radius of invert = ¼ width

Ratio of depth to height	Area = $(\text{width})^2 \times$ factor below	Wetted perimeter = width × factor below	Hydraulic radius = width × factor below
0.033	0.0102	0.332	0.0318
0.067	0.0280	0.464	0.0603
0.100	0.0496	0.580	0.0855
0.133	0.0734	0.685	0.1071
0.167	0.0982	0.785	0.1250
0.200	0.1347	0.937	0.1437
0.222	0.1551	1.012	0.1532
0.250	0.1862	1.106	0.1685
0.267	0.2049	1.159	0.1768
0.333	0.2840	1.375	0.2066
0.400	0.377	1.583	0.238
0.500	0.5091	1.892	0.269
0.600	0.655	2.195	0.298
0.667	0.7558	2.394	0.3157
0.700	0.8057	2.494	0.322
0.750	0.8795	2.646	0.3325
0.800	0.9503	2.806	0.338
0.900	1.0746	3.170	0.339
1.000	1.1485	3.965	0.2897

Simplified Kutter Formula.—Prof. H. E. Babbitt[1] has called attention to the fact that the omission of the term $0.0028/S$ in the Kutter formula will generally cause no greater differences in results, as compared with the complete formula, than those accompanying a difference of 0.001 in the value of n. This modification is, therefore, justified in some cases, and the work of computation may be greatly reduced, as compared with that required for the original formula.

Manning Formula.—Robert Manning[2] published a new and simpler formula for flow in open channels, in which the value of C in the Chezy formula is

$$C = \frac{1.486}{n} R^{1/6}$$

This form was adopted in order to use the same values of n as in Kutter's formula.

The Manning formula may also be written

$$v = KR^{2/3}S^{1/2}$$

in which

$$K = \frac{1.486}{n}$$

An extensive comparison of values of n, computed by the Kutter and Manning formulas from the results of experiments, was made by King,[3] from which he concludes that

> . . . the agreement between Manning's n and Kutter's n is most remarkable, and . . . the two formulas give results agreeing well within the limits of uncertainty which must exist in selecting the proper value of n, for all working conditions.

He also gives a table, here reproduced as Table 17, containing the computed values of Kutter's n and Manning's n[4] for identical values of C in the Chezy formula, which shows how closely they agree within the range of ordinary experience. It should be noted, however, that the differences are most marked when the values of R are small, and consequently the divergence between the results of the Manning and Kutter formulas would be more marked in the case of small than of large sewers, if identical values of n were used. In such a case, the use of the Kutter formula would result in the larger size of sewer.

[1] In *Eng. Contr.*, 1922; **57**, 128.

[2] *Trans. Inst. Civil Eng. Ireland*, 1890.

[3] "Handbook of Hydraulics," First Ed., pp. 197 and 200.

[4] And also of a similar factor in the Bazin formula, referred to hereinafter.

TABLE 17.—COMPARISON OF COEFFICIENTS OF ROUGHNESS IN KUTTER'S, MANNING'S AND BAZIN'S FORMULAS

King's "Handbook of Hydraulics"

Hydraulic radius R, feet	C, Chezy formula	n, Kutter's formula S = 0.000025	S = 0.00005	S = 0.0001	S = 0.0002	S = 0.0004	S = 0.001	S = 0.01	n, Manning's formula	m, Bazin's formula
0.1	10	0.040	0.042	0.044	0.045	0.047	0.049	0.050	0.101	4.67
	15	0.028	0.032	0.034	0.035	0.037	0.038	0.039	0.067	3.01
	20	0.022	0.025	0.027	0.028	0.029	0.031	0.032	0.051	2.18
	30	0.016	0.018	0.020	0.022	0.022	0.023	0.023	0.034	1.34
	40	0.013	0.015	0.016	0.017	0.018	0.019	0.019	0.025	0.930
	50	0.011	0.012	0.014	0.015	0.015	0.016	0.016	0.020	0.681
	75	.....	0.009	0.010	0.011	0.011	0.012	0.012	0.014	0.348
	100	.....	.....	.....	0.009	0.009	0.010	0.010	0.010	0.182
0.2	15	0.037	0.040	0.041	0.042	0.042	0.043	0.044	0.076	4.25
	20	0.029	0.032	0.034	0.036	0.037	0.038	0.039	0.057	3.08
	30	0.021	0.023	0.024	0.026	0.027	0.028	0 028	0.038	1.90
	40	0.017	0.018	0.020	0.021	0.022	0.022	0.023	0.028	1.31
	50	0.014	0.015	0.017	0.018	0.018	0.019	0.019	0.023	0.963
	75	0.010	0.011	0.012	0.013	0.013	0.014	0.014	0.015	0.492
	100	.....	0.009	0.010	0.010	0.011	0.011	0.011	0.011	0.258
	125	.....	.....	.....	0.009	0.009	0.009	0.009	0.009	0.117
0.4	20	0.038	0.040	0.042	0.045	0.045	0.046	0.046	0.064	4.35
	30	0.027	0.029	0.030	0.032	0.032	0.033	0.034	0.043	2.69
	40	0.021	0.022	0.024	0.025	0.026	0.026	0.027	0.032	1.86
	50	0.017	0.019	0.020	0.021	0.022	0.022	0.023	0.026	1.36
	75	0.012	0.013	0.015	0.015	0.016	0.016	0.016	0.017	0.696
	100	0.010	0.011	0.0115	0.012	0.012	0.013	0.013	0.013	0.364
	125	.....	0.009	0.010	0.010	0.010	0.010	0.011	0.010	0.165
	150	.....	.....	......	0.009	0.009	0.009	0.009	0.009	0.032
0.6	30	0.031	0.033	0.035	0.036	0.036	0.037	0.038	0.046	3.29
	40	0.024	0.026	0.027	0.028	0.029	0.030	0.030	0.034	2.28
	50	0.020	0.021	0.023	0.024	0.024	0.024	0.025	0.027	1.67
	75	0.014	0.015	0.016	0.017	0 017	0.017	0.017	0.018	0.853
	100	0.011	0.012	0.013	0.013	0.013	0.013	0.014	0.014	0.446
	125	0.009	0.010	0.010	0.011	0.011	0.011	0.011	0.011	0.202
	150	.....	.....	0.009	0.009	0.009	0.009	0.009	0.009	0.039
0.8	30	0.035	0.036	0.038	0.039	0.040	0.040	0.041	0.048	3.80
	40	0.027	0.028	0.030	0.031	0.031	0.031	0.032	0.036	2.63
	50	0.022	0.023	0.024	0.025	0.026	0.026	0.026	0.029	1.93
	75	0.015	0.017	0.017	0.017	0.018	0.018	0.018	0.019	0.985
	100	0.012	0.013	0.013	0.013	0.014	0.014	0.014	0.014	0.515
	125	0.010	0.010	0.011	0.011	0.012	0.012	0.012	0.0115	0.233
	150	.....	0.009	0.009	0.010	0.010	0.010	0.010	0.010	0.045
1.0	30	0.037	0.039	0.041	0.042	0.042	0.043	0.043	0.050	4.25
	40	0.029	0.030	0.031	0.033	0.033	0.034	0.034	0.037	2.94

TABLE 17.—COMPARISON OF COEFFICIENTS OF ROUGHNESS IN KUTTER'S, MANNING'S, AND BAZIN'S FORMULAS.—(*Continued*)

Hydraulic radius R, feet	C, Chezy formula	n, Kutter's formula							n, Manning's formula	m, Bazin's formula
		S = 0.000025	S = 0.00005	S = 0.0001	S = 0.0002	S = 0.0004	S = 0.001	S = 0.01		
1.0	50	0.024	0.025	0.026	0.027	0.027	0.028	0.028	0.030	2.15
	75	0.016	0.017	0.018	0.019	0.019	0.019	0.019	0.020	1.10
	100	0.013	0.014	0.014	0.014	0.015	0.015	0.015	0.015	0.576
	125	0.010	0.011	0.012	0.012	0.012	0.012	0.012	0.012	0.261
	150	0.009	0.009	0.010	0.010	0.010	0.010	0.010	0.010	0.050
1.5	30	0.043	0.044	0.045	0.046	0.047	0.047	0.048	0.053	5.20
	40	0.034	0.035	0.036	0.037	0.037	0.037	0.037	0.040	3.60
	50	0.027	0.029	0.029	0.030	0.030	0.030	0.031	0.032	2.63
	75	0.019	0.020	0.020	0.020	0.021	0.021	0.022	0.021	1.35
	100	0.014	0.015	0.015	0.016	0.016	0.016	0.016	0.016	0.705
	125	0.012	0.012	0.012	0.013	0.013	0.013	0.013	0.013	0.319
	150	0.010	0.010	0.011	0.011	0.011	0.011	0.011	0.011	0.061
2.0	40	0.037	0.038	0.039	0.040	0.040	0.040	0.040	0.042	4.16
	50	0 030	0.031	0.032	0.032	0.032	0.033	0.033	0.033	3.04
	75	0.021	0.022	0.022	0.022	0.022	0.022	0.022	0.022	1.56
	100	0 016	0.016	0.016	0.017	0.017	0.017	0.017	0.017	0.814
	125	0.013	0.013	0.013	0.013	0.013	0.013	0.013	0.013	0.369
	150	0.011	0.011	0.011	0.011	0.011	0.011	0.011	0.011	0.071
	175	0.009	0.009	0.010	0.010	0.010	0.010	0.010	0.010	−0.114
3.0	40	0.043	0.043	0.044	0.044	0.044	0.044	0.044	0.045	5.09
	50	0.035	0.035	0.036	0.036	0.036	0.036	0.036	0.036	3.73
	75	0.024	0.024	0.024	0.024	0.024	0.024	0.024	0.024	1.91
	100	0.018	0.018	0.018	0.018	0.018	0.018	0.018	0.018	0.998
	125	0.014	0.014	0.014	0.014	0.014	0.014	0.014	0.014	0.452
	150	0.012	0.012	0.012	0.012	0.012	0.012	0.012	0.012	0.087
	175	0.010	0.010	0.010	0.010	0.010	0.010	0.010	0.010	−0.173
4.0	50	0.039	0.039	0.039	0.039	0.039	0.039	0.039	0.037	4.30
	75	0.026	0.026	0.025	0 025	0.025	0.025	0.025	0.025	2.20
	100	0.019	0.019	0.019	0.019	0.019	0.019	0.019	0.019	1.15
	125	0.015	0 015	0.015	0.015	0.015	0.015	0.015	0.015	0.522
	150	0.013	0.013	0.012	0.012	0.012	0.012	0.012	0.012	0.100
	175	0.011	0.011	0.011	0.011	0.011	0.011	0.011	0.011	−0.200
	200	0.009	0.009	0.009	0.009	0.009	0.009	0.009	0.009	−0.424
6.0	50	0.045	0.045	0.044	0.043	0.042	0.041	0.041	0.040	5.27
	75	0.030	0.029	0.029	0.028	0.027	0.027	0.027	0.027	2.70
	100	0.022	0.021	0.021	0.020	0.020	0.020	0.020	0.020	1.41
	125	0.017	0.017	0.016	0.016	0.016	0.016	0.016	0.016	0.639
	150	0.014	0.014	0.013	0.013	0.013	0.013	0.013	0.013	0.122
	175	0.012	0.012	0.011	0.011	0.011	0.011	0.011	0.011	−0.245
	200	0.010	0.010	0.010	0.010	0.010	0.010	0.010	0.010	−0.519

TABLE 17.—COMPARISON OF COEFFICIENTS OF ROUGHNESS IN KUTTER'S, MANNING'S, AND BAZIN'S FORMULAS.—(*Continued*)

Hydraulic radius R, feet	C, Chezy formula	n, Kutter's formula							n, Manning's formula	m, Bazin's formula
		S = 0.000025	S = 0.00005	S = 0.0001	S = 0.0002	S = 0.0004	S = 0.001	S = 0.01		
8.0	50	0.048	0.048	0.047	0.046	0.045	0.044	0.044	0.042	6.09
	75	0.033	0.031	0.030	0.029	0.028	0.028	0.028	0.028	3.11
	100	0.024	0.023	0.022	0.021	0.021	0.020	0.020	0.021	1.63
	125	0.019	0.018	0.017	0.017	0.016	0.016	0.016	0.017	0.738
	150	0.015	0.014	0.014	0.014	0.013	0.013	0.013	0.014	0.141
	175	0.013	0.012	0.012	0.011	0.011	0.011	0.011	0.012	−0.283
	200	0.011	0.010	0.010	0.010	0.010	0.010	0.010	0.010	−0.600
10.0	75	0.039	0.034	0.032	0.031	0.030	0.030	0.030	0.029	3.48
	100	0.027	0.024	0.023	0.022	0.022	0.021	0.021	0.022	1.82
	125	0.019	0.018	0.018	0.017	0.017	0.017	0.016	0.017	0.825
	150	0.016	0.015	0.014	0.014	0.014	0.014	0.014	0.015	0.158
	175	0.013	0.013	0.012	0.012	0.012	0.012	0.011	0.012	−0.316
	200	0.011	0.011	0.010	0.010	0.010	0.010	0.010	0.011	−0.670
	225	0.010	0.010	0.009	0.009	0.009	0.009	0.009	0.010	−0.949
20.0	75	0.045	0.041	0.037	0.036	0.034	0.033	0.033	0.033	4.92
	100	0.033	0.029	0.026	0.025	0.024	0.023	0.023	0.024	2.58
	125	0.024	0.021	0.020	0.019	0.018	0.018	0.018	0.020	1.17
	150	0.019	0.017	0.016	0.015	0.015	0.014	0.014	0.016	0.224
	175	0.016	0.014	0.013	0.012	0.012	0.012	0.012	0.014	−0.447
	200	0.013	0.012	0.011	0.011	0.010	0.010	0.010	0.012	−0.948
	225	0.011	0.010	0.010	0.009	0.009	0.009	0.009	0.011	−1.34
30.0	75	0.050	0.047	0.041	0.039	0.036	0.035	0.034	0.035	6.03
	100	0.037	0.031	0.028	0.026	0.025	0.024	0.024	0.026	3.16
	125	0.027	0.023	0.021	0.019	0.019	0.018	0.018	0.021	1.43
	150	0.022	0.018	0.016	0.015	0.015	0.015	0.015	0.018	0.274
	175	0.017	0.015	0.013	0.013	0.012	0.012	0.012	0.015	−0.548
	200	0.014	0.012	0.011	0.011	0.011	0.010	0.010	0.013	−1.16
	225	0.012	0.011	0.010	0.010	0.009	0.009	0.009	0.012	−1.64

Solution of the Manning Formula.—The form of the Manning formula is such that it may readily be solved with the aid of a table of logarithms. It is advantageous, however, to make use of tables or diagrams when many computations are to be made. Such tables or diagrams are simpler in form than those required for Kutter's formula.

A simple form of diagram for the general solution of Manning's formula is that given in King's "Handbook of Hydraulics," and here reproduced as Fig. 25. The method of using this diagram is described below the figure.

Straight-line diagrams for Manning's formula may be prepared by the use of logarithmic cross-section paper, and several such diagrams

have been prepared. One of the best is the nomographic chart devised by Prof. Elmo G. Harris.[1]

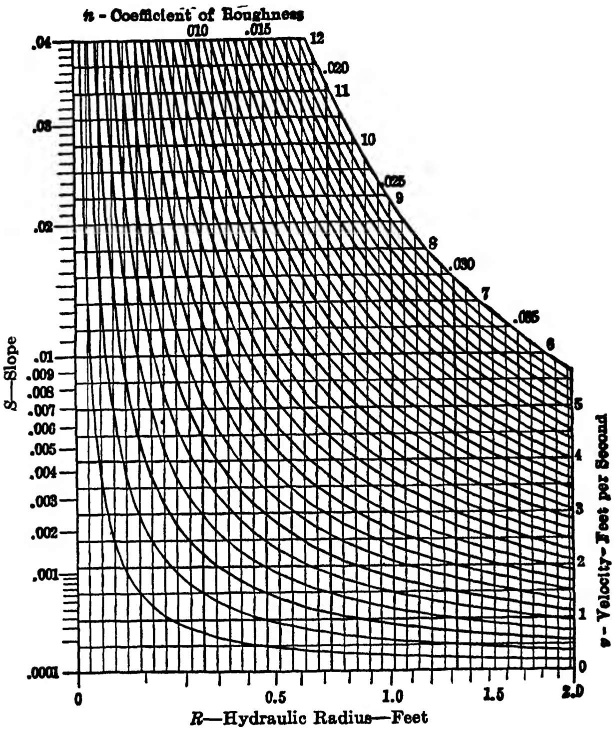


FIG. 25.—Diagram for solution of Manning's formula $v = \frac{1.486}{n} R^{2/3} S^{1/2}$ for sewers and small canals. (*King's Handbook of Hydraulics.*)

Any three quantities known, to find the unknown. Taking the equation in the form $vn = 1.486R^{2/3}S^{1/2}$, find the intersection of the two known quantities which occur on the same side of the equation, and follow the curved guide lines to the intersection with the third known quantity. The other coordinate of this intersection gives the quantity desired.

Bazin Formula.—H. Bazin published[2] in 1897 a formula for C in the Chezy formula, which is:

$$C = \frac{157.6}{1 + \frac{m}{\sqrt{R}}}$$

This superseded an earlier formula of Bazin. It is widely used in Europe, particularly in France, and has become sufficiently known in this country so that most detailed analyses of experimental results

[1] *Eng. News-Rec.* 1920; **85**, 837.
[2] "Annales des Ponts et Chaussées," 1897.

include values of m, as well as n for Kutter's and Manning's formulas. Table 17 includes the values of m corresponding to various values of C, R and S.

Table 18, from King's "Handbook of Hydraulics," gives values of m suitable for use with various kinds of conduits.

TABLE 18.—VALUES OF m FOR BAZIN'S FORMULA

	Best	Good	Fair	Bad
Vitrified sewer pipe	0.10	0.40	0.60	0.90
Common clay drain tile	0.20	0.30	0.50	0.90
Glazed brickwork	0.10	0.25	0.40	0.60
Brick in cement mortar	0.25	0.40	0.60	0.90
Neat cement surfaces	0.00	0.10	0.25	0.40
Cement-mortar surfaces	0.10	0.20	0.40	0.60
Concrete pipe	0.25	0.40	0.60	0.75
Plank flumes, planed	0.00	0.25	0.40	0.50
Plank flumes, unplaned	0.10	0.40	0.50	0.60
Plank flumes, with battens	0.25	0.60	0.75	1.00
Concrete-lined channels	0.25	0.50	0.75	1.00
Rubble masonry	0.90	1.25	1.90	2.50
Dry rubble	1.90	2.50	2.90	3.15
Ashlar masonry	0.40	0.50	0.65	0.90
Smooth metal flumes	0.10	0.25	0.40	0.60
Corrugated metal flumes	1.60	1.90	2.20	2.50
Earth canals in good condition	0.90	1.25	1.60	1.90
Earth canals with weeds, rocks, etc	1.90	2.50	3.15	3.80
Canals excavated in rock	2.50	3.15	3.70	4.20
Natural streams in good condition	1.90	2.50	3.15	3.80
Natural streams with weeds, rocks, etc	3.15	4.40	6.30	8.80

Hazen and Williams' Formula.—Of late years, several exponential formulas for the flow of water in pipes and conduits have been developed. Of these the most important is that developed in 1902 by Allen Hazen and Gardner S. Williams, which agrees closely with observed results and has the great merit that it can be applied with facility through the special slide rule designed and graduated for the solution of problems by it. Tables have also been prepared covering its application.[1] Inasmuch as careful comparison of this formula has been made with the better-known Kutter's formula, and as the use of the slide rule is not only convenient but effects a very considerable saving in time in making many hydraulic computations, this formula is of particular

[1] WILLIAMS and HAZEN, "Hydraulic Tables," Third Edition, 1920.

importance. While this formula has had application most often to pipes discharging under pressure, it may also be used in sewer computations.

The Hazen and Williams formula is

$$v = C_h R^{0.63} S^{0.54} 0.001^{-0.04}$$

The authors of the formula say of it,[1]

The exponents in the formula used were selected as representing as nearly as possible average conditions, as deduced from the best available records of experiments upon the flow of water in such pipes and channels as most frequently occur in water-works practice. The last term, $0.001^{-0.04}$, is a constant, and is introduced simply to equalize the value of C_h with the value in the Chezy formula, and other exponential formulas which may be used, at a slope of 0.001 instead of at a slope of 1.

This formula may also be written (since $0.001^{-0.04} = 1.318$),

$$v = 1.318\, C_h R^{0.63} S^{0.54}$$

With regard to the coefficients to be used in this formula in general design, Williams and Hazen suggest the following values for C_h:

140 for new cast-iron pipe when very straight and smooth
130 for new cast-iron pipe under ordinary conditions
100 for old cast-iron pipe under ordinary conditions, this value to be used for ordinary computations anticipating future conditions

110 for new riveted steel pipe
95 for steel pipe under future conditions

140 for new lead, brass, tin, or glass pipe with very smooth surface
130 to 120 ditto, when old
120 for smooth wooden pipe or wooden stave pipe

140 for the masonry conduits of concrete or plaster with very smooth surfaces and when clean
130 ditto, after a moderate time when slime-covered
120 ditto, under ordinary conditions

110 for cement-lined pipe (Metcalf; not given by Williams & Hazen)
100 for brick sewers in good condition
110 for vitrified pipe sewers in good condition

[1] Williams and Hazen, "Hydraulic Tables," pp. 1 and 2.

The Hazen and Williams formula reduces to the following forms for the given values of C_h

$$
\begin{aligned}
&\text{when } C_h = 100, \; v = 131.8R^{0.63}S^{0.54} = 55.0D^{0.63}S^{0.54}\\
&\text{when } C_h = 110, \; v = 145.0R^{0.63}S^{0.54} = 60.5D^{0.63}S^{0.54}\\
&\text{when } C_h = 120, \; v = 158.2R^{0.63}S^{0.54} = 66.0D^{0.63}S^{0.54}\\
&\text{when } C_h = 130, \; v = 171.4R^{0.63}S^{0.54} = 71.6D^{0.63}S^{0.54}\\
&\text{when } C_h = 140, \; v = 184.6R^{0.63}S^{0.54} = 77.1D^{0.63}S^{0.54}
\end{aligned}
$$

The ratio of the values of v, S, and D for other values of C_h to their value for $C_h = 100$ and $C_h = 130$ are shown in Fig. 26.[1]

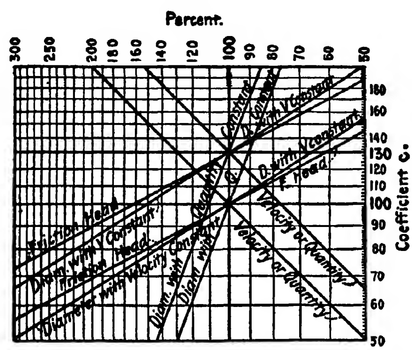


FIG. 26.—Relations between factors in Hazen-Williams formula.

Figures 27 and 28 show the discharges of pipes of various sizes according to the Hazen and Williams formula for $C_h = 130$ and $C_h = 100$, respectively.

The relation between the value of C_h in the Hazen and Williams formula and the C of the Chezy formula may be found by equating the value of S in these two formulas, which gives the equation

$$C \text{ (Chezy)} = 1.1506\, C_h^{0.9259} v^{0.0741} D^{0.0833}$$

NONUNIFORM FLOW

Conditions of steady nonuniform flow exist when a constant quantity of water flows with variable cross-sections, slopes, and velocities. The surface of the water is, therefore, not parallel to the invert of the conduit. This condition always exists at points of changing equilibrium, such as at and near changes in grade and in cross-section, and above

[1] For further discussion of the Hazen and Williams formula, see *Eng. Rec.* 1903; **47**, 667.

obstructions or free outlets.[1] Typical examples of nonuniform flow are shown in Figs. 29 to 31.

General Equation for Nonuniform Flow.—Assume a reach of conduit short enough so that the loss due to friction may be computed with

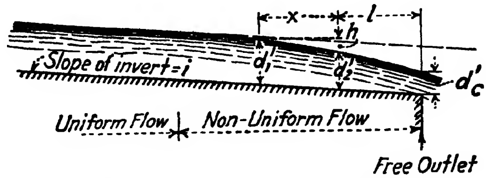


FIG. 29.—Drawdown curve, free discharge.

sufficient accuracy by one of the formulas for uniform flow, such as Manning's formula, making use of the mean hydraulic radii and velocities. Then the average velocity in the reach would be

$$v = \frac{1.486}{n}R^{2/3}S^{1/2}$$

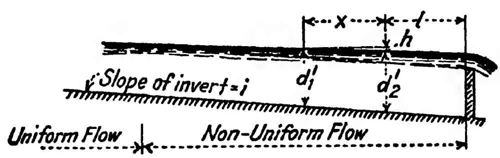


FIG. 30.—Backwater curve.

Taking $v = \frac{1}{2}(v_1 + v_2)$ or the average between the velocities at the ends of the reach,

$$S = \frac{n^2(v_1 + v_2)^2}{8.83R^{4/3}}$$

Since the total fall in the water surface h is equal to the sum of the frictional loss and the difference in the velocity heads,

$$h = xS + \frac{v_2^2}{2g} - \frac{v_1^2}{2g}$$

[1] HINDS, JULIAN, "The Hydraulic Jump and Critical Depths in the Design of Hydraulic Structures," *Eng. News-Rec.*, 1920; **85**, 1034.

BABBITT, H. E., "Nonuniform Flow and Significance of Dropdown Curve in Conduits," *Eng. News-Rec.*, 1922; **89**, 1067.

HILL, C. D., "Application of the Dropdown Curve in Chicago Sewers," *Eng. News-Rec.*, 1923; **90**, 707.

HUSTED, ALVA G., "New Method of Computing Backwater and Dropdown Curves," *Eng., News-Rec.* 1924; **92**, 719.

WOODWARD, SHERMAN M., "Theory of the Hydraulic Jump and Backwater Curves," in *Tech. Rep.*, Part III, Miami Conservancy District.

The same expression for the drop in water surface is applicable on the basis of any formula for frictional loss.

If d'_1 and d'_2 represent the depths of water at the two ends of the reach, and i the inclination of the bottom, then

$$h = xi + (d'_1 - d'_2)$$

Equating the values of h,

$$x = \left[d'_1 + \frac{v_1^2}{2g} - \left(d'_2 + \frac{v_2^2}{2g}\right)\right] \div (S - i)$$

From this expression the distance x between any two sections of the stream in which the change in depth is $d'_1 - d'_2$ can be computed approximately.

The foregoing expressions are general and may be applied to any case of steady nonuniform flow, within the limit of the accuracy of the assumptions.

Critical Depth.—In the case of free discharge, illustrated in Fig. 29, or of a decided increase in inclination of the channel, as shown in Fig. 31, the depth of flow at the outlet, or at the break in grade, will be

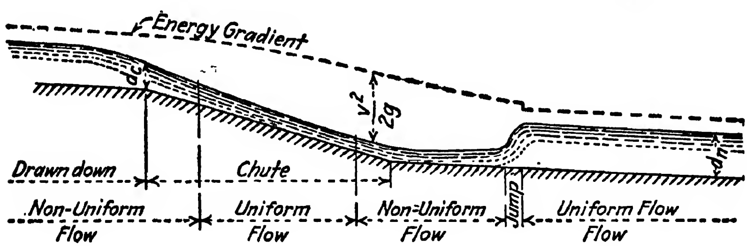


FIG. 31.—Drawdown, chute, and hydraulic jump.

definitely fixed by the rate of discharge, for any given conduit. The case is analogous to the discharge of a weir. This depth is called the *critical depth*, designated as d'_c, and is the depth for which $d' + \frac{v^2}{2g}$ is a minimum. Then for a rectangular section $d'_c + \frac{Q^2}{2gb^2(d_c)^2}$ is a minimum, whence

$$d'_c = \frac{Q^{2/3}}{g^{1/3}b^{2/3}} * \quad \text{or} \quad (d'_c)^3 = \frac{Q^2}{gb^2}$$

In the expression, for a given depth d'_c, Q varies directly with b, as would be expected for the rectangular section. It is, therefore, practicable to construct a table giving values of Q per foot of width for various values of d'_c. Such values are given in Table 19.

* For this case $v = \sqrt{gd'}$ and $v^2/2g = d'/2$ (velocity head = ½ depth).

TABLE 19.—THEORETICAL DISCHARGE PER FOOT OF WIDTH FOR VARIOUS VALUES OF "CRITICAL DEPTH" IN RECTANGULAR CHANNELS

$Q^{2/3} = d_c'g^{1/3}$ $\therefore Q = (d_c')^{3/2}g^{1/2}$. $g^{1/2} = 5.674$ (slide-rule computations)

Depth, d_c'	Discharge in cubic feet per second									
	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
0	0.0	0.18	0.51	0.94	1.44	2.00	2.64	3.33	4.07	4.86
1	5.70	6 58	7.50	8.45	9.43	10.4	11.5	12.6	13.7	14.8
2	16.0	17.3	18.5	19.8	21.1	22.4	23.8	25.2	26.6	28.0
3	29.5	31.0	32.5	34.0	35.1	37.1	38.7	40.4	42.0	43.7
4	45.4	47.1	48.8	50.6	52.4	54.2	56.0	57.8	59.7	61.6
5	63.5	65.5	67.4	69.4	71.3	73.3	75.3	77.3	79.4	81.5
6	83.5	75 6	87.8	89.9	92.0	94.2	96.4	98.5	100.6	102.8
7	105.0	107.2	109.5	111.8	114	116	119	121	123	126
8	128	131	133	136	138	140	143	146	148	151
9	153	156	158	161	164	166	169	171	174	176
10	179	182	185	188	190	193	196	199	202	205

Similar computations of the critical depths corresponding to various rates of discharge for other forms of cross-section can be made, but they are likely to be very complex, and it is usually simpler to obtain the values by successive approximations. Prof. H. E. Babbitt[1] has done this for circular conduits and obtained the values given in Table 20.

All these figures are based upon theoretical computations. No experimental determinations of their correctness have been made. It is believed, however, that actual values are not likely to vary from those computed in this way, by more than 10 per cent.

Drawdown.—The transition from a condition of uniform flow in a conduit to the discharge at a free outlet or the drop at a chute is accomplished by a gradual lowering of the surface similar to the surface curve above a weir. If the conduit is on a flat slope and the normal velocity is low, the difference between the normal depth and the "critical depth" will be considerable, the drawdown will be of consequence and the drawdown curve will extend upstream for a material distance. In such a case it may sometimes be possible to make a saving by reducing the size of the conduit and eliminating the drawdown, or by lowering the roof while leaving the width unchanged. If, however, the conduit is on such a steep slope that the velocity is high and the critical depth but little less than the normal depth, no material reduction in size of conduit can be made, and the drawdown will not be significant. In those cases, too, where there is possibility that the conduit may flow full under pressure at times, reduction in section for drawdown would be undesirable.

[1] *Eng. News-Rec.*, 1922; **89**, 1069.

TABLE 20.—THEORETICAL VALUES OF "CRITICAL DEPTH" IN FEET, IN CIRCULAR CONDUITS FOR VARIOUS RATES OF DISCHARGE
H. E. Babbitt in *Eng. News-Rec.*, 1922; **89**, 1069

Rate of discharge, cubic feet per second	Diameter of conduit, feet									
	1	2	3	4	5	6	7	8	9	10
0.2	0.20									
0.4	0.27									
0.6	0.33									
0.8	0.38									
1.0	0.41	0.33								
1.5	0.52									
2	0.60	0.48								
3	0.73									
4	0.84									
5	....	0.80	0.66	0.64						
10	....	1.12	0.99	0.92	0.87	0.80				
15	....	1.42	1.23	1.12						
20	....	1.64	1.44	1.28	1.19					
30	....	1.80	1.80	1.60	1.54					
40	....	1.94	2.07							
50	....	....	2.28	2.08	1.98	1.89	1.79	1.72	1.71	1.60
100	....	....	2.88	3.08	2.85	2.67	2.56	2.48	2.39	2.25
150	....	....	....	3.64	3.73					
200	....	....	....	3.92	3.98	3.87	3.68	3.56		
300	....	....	....	....	4.65	4.80	4.55	4.40	4.14	4.15
400	....	....	....	....	4.86	5.44	5.22	5.04		
500	....	....	....	....	....	5.69	5.81	5.68	5.49	5.30
600	....	....	....	....	....	....	6.37	6.40		
700	....	....	....	....	....	....	6.55	6.68	6.53	6.35
800	....	....	....	....	....	....	....	7.04		
900										
1,000	....	....	....	....	....	....	6.79	7.48	7.74	7.70
1,200	....	....	....	....	....	....	....	....	8.24	8.25
1,400	....	....	....	....	....	....	....	....	8.55	8.75
1,600	....	....	....	....	....	....	....	....	....	9.25

The distance from the control section (the point of "critical depth"), to which the drawdown extends, can be computed approximately by successive steps, utilizing the general equation for nonuniform flow.

For example, take the case of a rectangular conduit 10 ft. wide, with a slope of 0.001, with $n = 0.015$ and $Q = 250$ c.f.s., discharging freely, as in Figs. 29 and 31. Using Manning's formula, we find, for $S = 0.001$, when $d' = 4.0$, $Q = 214$, and when $d' = 5.0$, $Q = 288$; and by inter-

polating, when $Q = 250$, $d' = 4.5$, which will be the depth of flow when the effect of the drawdown is not felt. From Table 19, the critical depth d'_c will be 2.7 ft. The drawdown curve will be that portion of the water surface between depths of 4.5 and 2.7 ft. Its computation is shown in the following tabulation:

d'	A	p	R	v	$\frac{v^2}{2g}$	$d'+\frac{v^2}{2g}$	Average R	Average v	nv	S	$S-i$	Change in $d'+\frac{v^2}{2g}$	x
2.7	27	15.4	1.75	9.28	1.34	4.04							
							1.81	8.81	0.132	0.0035	0.0025	0.04	16
3.0	30	16.0	1.87	8.33	1.08	4.08							
							1.95	7.84	0.118	0.0025	0.0015	0.16	107
3.4	34	16.8	2.02	7.35	0.84	4.24							
							2.09	6.98	0.105	0.0019	0.0009	0.24	266
3.8	38	17.6	2.16	6.61	0.68	4.48							
							2.23	6.32	0.0095	0.0014	0.0004	0.27	675
4.2	42	18.4	2.28	5.97	0.55	4.75							
							2.33	5.76	0.00865	0.0011	0.0001	0.23	2,300
4.5	45	19.0	2.37	5.56	0.48	4.98							
Total length of drawdown													3,364

Backwater.—The surface curve assumed by the water when backed up by a dam or other obstruction is called the backwater curve (Fig. 30). It may be required to determine the amount by which the depth is increased at specified points, or the distance upstream to which the effect of backwater can be detected.

For example, take a conduit of the same dimensions and character as in the example under "drawdown," but where the discharge is into a body of water with its surface 7.0 ft. above the bottom of the conduit. The procedure is exactly the same as in the example referred to, but the increments of depth are negative, and the value of $S - i$ is also negative. The computation for the case is as follows:

d'	A	p	R	v	$\frac{v^2}{2g}$	$d' + \frac{v^2}{2g}$	*Average* R	*Average* v	nv	S	$S - i$	Change in $d' + \frac{v^2}{2g}$	x
7.0	70	24	2.91	3.57	0.20	7.20							
							2.87	3.71	0.0557	0.00033	−0.00067	−0.47	700
6.5	65	23	2.82	3.85	0.23	6.73							
							2.77	4.01	0.0603	0.00043	−0.00057	−0.46	807
6.0	60	22	2.72	4.17	0.27	6.27							
							2.67	4.36	0.0655	0.00054	−0.00046	−0.45	978
5.5	55	21	2.62	4.55	0.32	5.82							
							2.56	4.77	0.0717	0.00067	−0.00033	−0.43	1,300
5.0	50	20	2.50	5.00	0.39	5.39							
							2.44	5.28	0.0793	0.00086	−0.00014	−0.41	2,920
4.5	45	19	2.37	5.56	0.48	4.98							
										Total extent of backwater			6,705 ft.

Chute.—A chute is a channel with so steep a grade that uniform flow can take place at a depth less than the "critical depth" (Fig. 31). The computation of flow in a chute, in so far as such flow is uniform, is accomplished by the use of the ordinary formulas, such as Kutter's or Manning's, although the applicability of these formulas at very high velocities is doubtful. Chutes are, however, usually short, and the portion of their length in which uniform flow conditions exist is often insignificant. Non-uniform flow obtains at the upper end of the chute, and the "hydraulic jump" may occur at the lower end if conditions beyond the chute are such as to produce it.

Hydraulic Jump (Fig. 31).—When water moving at a high velocity in a comparatively shallow stream strikes water having a substantial depth, there is likely to be a rise in the water surface of the stream, forming what is called "the hydraulic jump." The jump cannot occur unless the primary depth of the water is less than the critical depth, and when it does occur the water surface rises from a point below to a related point above that representing the critical depth. The depth d'_2 of water which will cause the swift-moving stream to form the jump may be computed from the equation

$$\frac{d'_1 d'_2(d'_1 + d'_2)}{2} = \frac{q^2}{g} = d'^3_c$$

The derivation of this relation is given below.

Let q equal the flow in 1 ft. of width of channel.

Let d'_1 and v_1, d'_2 and v_2, d'_c and v_c be, respectively, the depth and velocity in the channel immediately above the jump, below the jump, and at the point of critical depth. As shown on p. 102,

$$d'^3_c = \frac{q^2}{g} \quad \text{or} \quad q = \sqrt[2]{g d'^3_c}$$

but $q = v_1 d'_1 = v_2 d'_2$.

The change in velocity in passing through the jump is $v_1 - v_2$. The mass of water passing in 1 sec. is wq/g and the change in momentum is

$$\frac{wq}{g}(v_1 - v_2) = \frac{wq}{g}\left(v_1 - v_1\frac{d'_1}{d'_2}\right) = \frac{wqv_1(d'_2 - d'_1)}{gd'_2}$$

The static pressure upon the cross-section of the stream is $wd'^2_1/2$ above the jump, and $wd'^2_2/2$ below the jump. The difference is

$$\frac{wd'^2_2}{2} - \frac{wd'^2_1}{2} = \frac{w}{2}(d'^2_2 - d'^2_1).$$

The difference in static pressure represents a force acting in the opposite direction to the flow, and this is the force which causes the

change in momentum represented by the reduction in velocity from v_1 to v_2.

Therefore,

$$\frac{w}{2}(d_2'^2 - d_1'^2) = \frac{wqv_1}{g}\frac{(d_2' - d_1')}{d_2'}$$

Whence

$$d_2'^2 + d_2'd_1' = \frac{2qv_1}{g}$$

substituting $\frac{q}{d_1'}$ for v_1,

$$d_2'^2 + d_2'd_1' = \frac{2q^2}{gd_1'}$$

Whence

$$d_1'd_2'\frac{(d_1' + d_2')}{2} = \frac{q^2}{g}$$

which has been shown to be equal to $(d_c)^3$.

For example, take the case of 250 c.f.s. flowing in a rectangular conduit 10 ft. wide, with $n = 0.015$, as before; assume the inclination of the upper section to be 0.05; using Manning's formula (assumed applicable to very high velocities) the discharge for depth of 1.0 ft. is found to be 196 c.f.s., and for depth of 1.5 ft. 364 c.f.s.; and, by interpolation, the depth of flow in the upper section (chute) is fixed at 1.2 ft., which is less than the critical depth of 2.7 ft.

The normal depth of flow in the lower section with its slope of 0.001 is 4.5 ft. Assuming that this is the depth below the jump, the depth above the jump may be computed from the formula above as follows:

$$4.5d_1'\frac{(4.5 + d_1')}{2} = \frac{25^2}{32.2}$$

$$(d_1')^2 + 4.5d_1' = \frac{1,250}{144.9} = 8.64$$

$$(d_1')^2 + 4.5d_1' + 5.06 = 13.70$$

$$d_1' + 2.25 = 3.7$$

$$d_1' = 1.45$$

When the flow reaches the bottom of the chute, the flatter slope retards the flow and the depth increases. Assuming that the chute has sufficient length for the depth of flow to decrease from the critical depth at the top of the chute to approximately the normal depth of 1.2 ft. at the foot of the chute, the flow must continue in the lower section until the retardation has caused an increase in the depth of flow from 1.2 to 1.45 ft., or, say, 1.5 ft. The distance over which the flow must continue before the jump will occur is shown by the following computation.

Depth d'	Area A	p	R	v	$\frac{v^2}{2g}$	$d' + \frac{v^2}{2g}$	Average R	Average v	nv	S	$S - i$	Change in $d' + \frac{v^2}{2g}$	x
1.2	12	12.4	0.97	20.8	6.73	7.93							
							1.00	20.0	0.300	0.0408	0.0398	0.90	22.6
1.3	13	12.6	1.03	19.2	5.73	7.03							
							1.06	18.5	0.278	0.0325	0.0315	0.70	22.2
1.4	14	12.8	1.09	17.8	4.93	6.33							
							1.120	17.2	0.258	0.0258	0.0248	0.49	19.8
1.5	15	13.0	1.15	16.7	4.34	5.84							
							Total distance below chute						64.6

Then the jump will be located about 65 ft. below the end of the chute. It is to be noted that under some conditions of channel cross-section and slope, the length of the lower section may be insufficient for the water surface to rise to the required depth. It should also be noted that if the cross-section of the lower section differs from that of the chute so that the critical depth is below the depth of flow after it reaches the lower section, no jump will occur.

LOSSES OF HEAD FROM CAUSES OTHER THAN FRICTION

Velocity Head.—Strictly speaking, the head required to produce velocity, $v^2/2g$, is not lost, as it merely corresponds to the transformation of potential energy into kinetic energy, and theoretically it should be recoverable when the velocity is checked. This is the case, to a considerable extent, with closed pipes under pressure, but, in open-channel conditions, it is less common to recover a material part of the velocity head, except in the "hydraulic jump." It is usually advisable in sewerage work to consider the head used in producing velocity as lost head, and it is necessary to provide, in every case, sufficient head to develop velocity, in addition to that required to overcome friction and other resistances.

Entry Head.—In the case of a pipe leading from the side of a tank, the entry head loss has been found to be $0.505\frac{v^2}{2g}$, and is in addition to the velocity head. This is for the case where the corners are square and sharp. By suppressing or reducing the contraction resulting from the orifice, as by rounding the corners, the entry loss may be materially reduced, perhaps to $0.1\frac{v^2}{2g}$.

In the case of an open channel or conduit leading from a reservoir, the conditions are similar, but experimental data are lacking. It is probably safe to estimate that the loss will not be greater than that in a closed pipe.

Sudden Reduction of Cross-section.—In addition to the difference in velocity heads $\frac{v_2^2 - v_1^2}{2g}$, there is a further loss due to the resistance resulting from the disturbed flow conditions. Experiments on pipes have shown this additional loss to range from almost nothing to approximately $0.5\frac{v^2}{2g}$, (v being the velocity in the smaller pipe), depending on the ratio between the diameters, the coefficient being larger as the difference in diameter increases. There are no experimental data for open channels, but it seems reasonable to assume that similar relations exist.

Sudden Enlargement of Cross-section.—Archer has shown[1] that experimental data on losses due to sudden enlargements of closed pipes may be expressed by the formula

$$\text{loss} = 1.098\frac{v_1^{1.919}}{2g}\left(1 - \frac{A_1}{A_2}\right)^{1.919}$$

King and Wisler's "Hydraulics" contains a table (p. 161) showing that this is equivalent to amounts ranging from nearly nothing to the velocity head in the smaller pipe ($v_1^2/2g$).

Other disturbances to flow are caused by bends and partly closed valves in pipes; and by changes in direction, piers, side inlets, bulkheads, and the effect of wind upon the free surface of the liquid in open channels.

Valves.—Many experiments upon the loss resulting from partly closed valves in pipes indicate that in general the loss ranges from $0.2v^2/2g$ to $13.5\,\frac{v^2}{2g}$ (where v is the velocity in the pipe), as the ratio of the area of opening to that of the pipe decreases from 0.9 to 0.1, being about $2.7v^2/2g$ when the area of the opening is half that of the pipe.

Curves.—Many experiments upon loss of head due to curves in pipes have been made. Most of them, however, have been upon changes in direction amounting to 90 deg. Experiments in which the deflection differed from 90 deg. are too few to warrant definite conclusions, except for conditions similar to those of the experiments. In general, it appears probable that even a slight change in direction produces a condition of disturbed flow which increases the frictional resistance, and that the length in which such disturbed conditions exist is of greater significance than the sharpness of the deflection; in other words, a curve of short radius and correspondingly short length of curve is likely to result in smaller loss of head resulting from curvature than a curve of long radius with the same change of direction. There are limits, however, beyond which the opposite effect is probable.

The same conclusions relative to the effect of curves in open channels seem to be justified. H. P. Eddy has shown[2] that none of the formulas which have been proposed for loss due to curvature is applicable to open-channel conditions, and that such fragmentary data as are available for those conditions indicate that the effect of curvature has generally been equivalent to an increase in the value of n by an amount varying from 0.003 to 0.005 in the sections containing much curvature. In the present state of our knowledge, it seems logical to allow in design for the effect of curvature by a change in the coefficient of roughness.

[1] *Trans. Am. Soc. C. E.*, 1913; **76**, 999.
[2] *Eng. News-Rec*, 1921; **87**, 516.

Eddy's table, on which the foregoing conclusion is based, is as follows:

TABLE 21.—EFFECT OF CURVATURE UPON VALUE OF n IN OPEN CHANNELS, CALCULATED FROM EXPERIMENTAL DATA

Channel (concrete lined)	Value of n		Increase due to curvature	Radii of curves, feet	Velocity of flow, feet per second
	On tangent	On curves[1]			
Umatilla	0.0135 to 0.0137	0.0176 to 0.0184	0.0039 to 0.0049	50	7
		0.0162 to 0.0169	0.0025 to 0.0034	100	7
		0.0160 to 0.0173	0.0023 to 0.0038	250	7
Sulphur Creek	0.0108	0.0140	0.0032	2,865	20
Ridenbaugh	0.0121	0.0145	0.0024	955	3.7
North Canal (Central Oregon)	0.0177	0.0202	0.0025	410	3.0
	0.0176	0.0205	0.0029	and	2.9
	0.0192	0.0222	0.0030	383	2.1

[1] Computed.

Experiments by Scobey in 1924 upon the 60-in. reinforced-concrete pipe aqueduct of Tulsa, Okla., are of some significance in this connection, and may be considered as confirming the conclusions above, at least in part. No direct comparison is possible, partly because the aqueduct is a closed pipe under pressure, and partly because the changes in direction are angular bends, not curves. These bends were formed by butting together two sections of straight pipe at the angle required, and pouring concrete over a reinforcing cage at the joint, smoothing off the interior surface to a sharp curve at the outside of the bend. The deflections ranged from less than 5 deg. to about 28 deg.

Two sections were tested, one 80,898 ft. long, nearly straight, (containing 6 bends with a total deflection of 52.3 deg.), the other 34,788 ft. long, containing 29 bends with a total deflection of 455.7 deg. The velocity was 2.25 ft. per second. The value of n in the two sections were 0.0107 and 0.0111, showing an increase of 0.0004 for the section containing the greater changes in direction. In this section the average distance between bends was 1,200 ft. Assuming the effect of the bends in causing disturbed flow conditions to extend 100 ft. below the bends, one-twelfth of the total length of the section was so affected, and the increased loss in this portion would correspond to an increase in n of 12 times 0.0004, or 0.0048,[1] a figure comparable to those tabulated in Table 21.

In his discussion of the Tulsa experiments, Scobey has computed the average loss of head per degree of deflection as 0.00138 ft., and the average per bend, irrespective of the deflection, as 0.0226 ft., for a velocity of 2.25 ft. per second. The extent to which these figures are applica-

[1] Based on the assumption that loss of head varies directly with n, which is nearly correct for the small variations here considered.

ble to other situations, particularly to flow in open channels, is doubtful, but the significant data are so few that nothing which may be helpful should be left out of consideration.

"BANKING" ON CURVES

In some cases, the banking or superelevation of the water surface along the outer wall of a curved channel may be an item of considerable importance. This is especially the case with stream channels, open flumes, or flat-topped conduits in which it is desirable that the water should not touch the roof.

The excess in elevation of the water at the outer bank may be computed approximately by the formula

$$E = \frac{v^2 b}{gr}$$

in which E represents the difference in elevation of water surface at the two banks, v is the average velocity in the cross-section, b is the breadth of the channel or stream, and r is the radius of the center line of the channel.

It has been found in some cases that the actual difference in elevation is slightly greater than would be given by this formula.

CHAPTER III

VELOCITIES AND GRADES

Distribution of Velocity in Cross-section.—The moving body of water in any conduit travels at an average velocity v which is the mean of the velocities of all the filaments in a cross-section. These velocities are not uniform, but are least for the filaments in contact with the enclosing walls of the conduit. In a straight, closed conduit or pipe having a perimeter of uniform character, where the frictional effect of the walls is everywhere the same, the maximum velocity will be found at the center of the cross-section, or in the axis of the conduit. In an open conduit, the maximum velocity will be found at the greatest possible distance from all surfaces against which there is friction; the greater the friction upon any surface, the greater will be the distance of the point of maximum velocity from that surface. The water surface, or plane of contact between the flowing water and the atmosphere, is not a frictionless surface (except when the air is moving in the same direction at the same velocity), and, consequently, the maximum velocity is below the surface, but at a much less distance than from the walls of the enclosing channel.

Figure 13 shows the variation in velocity of flow in two Boston Metropolitan sewers. The amounts by which the curves vary from regular and smooth curves indicate the extent of disturbing influences of some kind, provided the measurements were free from errors.

An examination of the results of a large number of determinations of variation of velocity in the cross-section of pipes and open channels has shown that, in general, the curve of variation of velocity in any axial section of a closed pipe, or vertical section in an open channel, is approximately parabolic in form, with the axis of the parabola approximately in the thread of maximum velocity. The filaments in contact with the surface of the conduit are moving at velocities considerably less than the mean velocity in the cross-section.

Ratio of Mean to Maximum Velocity.—In straight closed pipes, the mean velocity has been found to be about 0.85 of the maximum velocity in the cross-section (the center velocity) and to be located about three-fourths of the radius from the center. The velocity at the perimeter is about one-half the center velocity.

In open channels, the mean velocity *in any longitudinal vertical section* is usually between 0.80 and 0.95 of the surface velocity. The

thread of mean velocity is usually between 0.55 and 0.65 of the depth below the surface, and the average of the velocities at 0.2 and 0.8 depth usually gives the mean velocity in the vertical plane within about 2 per cent.

The ratio of the mean velocity in the cross-section to the maximum surface velocity is not a constant, but it has usually been found to lie between 0.70 and 0.85 in all types of open channels, and the value 0.80 has often been used as a rough approximation.

MINIMUM VELOCITIES AND GRADES

Velocity and Transporting Power of Water.—The transporting capacity of water, due to its velocity, plays an important part in the disposal of sewage by dilution and diffusion and in preventing clogging of sewers and local formation of sludge banks as a result of the settling out of the heavier particles of sewage. The prevention of clogging of sewers is discussed hereafter under Self-cleaning Velocities in Sewers. It has been shown[1] that the transporting capacity of water varies as the sixth power of its velocity, so that if the velocity be doubled the transporting capacity is sixty-four times as great. Therefore, any influence which tends to check the velocity at any point immediately results in substantial reduction of its carrying capacity, and the subsequent deposition of particles which had been carried along readily by the current of greater velocity. The form and adhesive quality of the particles also plays a part in the formation of sludge banks.

Much of the information relating to the transporting capacity of streams is almost valueless, owing to the lack of exact knowledge of the velocity near the bottom of the stream,[2] which, together with the character of the material composing the bottom and the depth and, hence, the pressure of water upon it, are the most important elements in the problem of erosive action. Comparisons with average velocities are of slight significance. Freeman has called pointed attention to these facts in his Charles River Dam report, in which he cited the opinion of the veteran engineer, Hiram F. Mills, in regard to the misuse of the observations of Dubuat, who made experiments in 1780 upon the capacity of a stream in a wooden trough to move particles on its bottom. All of these observations failed to take into account the varying velocities of flow in any vertical section. Many engineers who have made use of the results of those experiments have failed to recognize this fact, as well as the effect of the character of the material, the coating of slime or

[1] MERRIMAN, "Hydraulics," Ninth Edition, 340.

[2] In general, the velocity near the bottom of an open channel is between 40 and 80 per cent of the mean velocity, depending largely upon the roughness of the bottom; two-thirds of the mean velocity may be taken or a rough approximation to the bottom velocity in sewers.

colloidal surface which forms upon the bottom and the effect of the pressure upon the material, due to the depth of water. Freeman quotes observations made by Mills and Hale on the Essex Company's canal adjoining the Merrimac River in Lawrence, which were made with sufficient care to be significant, using a current meter to determine the distribution of velocities.

At Station 1, middle of west chord of Everett Railroad bridge:

Banks and bed completely and smoothly covered with fine sand, as per sample, whose mechanical analysis is given in table following. Deposit 8 to 12 in. deep. Surface near the bottom marked with little waves of sand ⅜ in. high, probably rolled up by the more rapid velocity when emptying canal. Side slopes smooth and free of wave marks. Sand so soft and so like quicksand that one's feet sink into it 3 in. while walking across, or, when standing still for a minute or two, the feet gradually sink into it about 8 to 12 in. This sand plainly is not being scoured, although it is softer than any silt that I have seen uncovered at low tide on the shores of Boston harbor, except perhaps the silty sludge in immediate proximity to certain sewers.

	Ft. per second
Maximum surface velocity in center found to be	1.3
Mean velocity of center section	1.0
Velocity at 3 in. from bottom	0.8

This shows that a particularly soft bottom was not eroded by a bottom velocity of about 0.8 ft. per second, and that the condition was one that favored deposits.

Station 2, at upstream side of Union Street bridge:

General appearance the same as at Station 1, except that surface of sand in deepest portion of canal is covered by sand waves averaging about 1 in. high, with crests transverse to current, suggesting a rolling along of the sand grains which perhaps has been induced by the higher velocity from drawing off and refilling the canal a few times very recently, rather than by the ordinary flow. I find, on tramping back and forth over the silt, that it is much more firm than at Station 1.

	Ft. per second
Maximum surface velocity	1.9
Mean velocity of center section	1.5
Velocity at 3 in. from bottom in middle	1.2

With these velocities, silt of this quality is deposited 12 in. deep, and apparently is rolled into waves only by the recent drawing off of canal, since no sand waves are found more than halfway up on the sloping sides of canal. The indication is that a bottom velocity of 1.2 ft. per second favors deposit and not scour.

Station 3, from same cross-section, but about three-quarters distance up slope from center toward north side and 6 or 8 ft. up from bottom level, where there were no sand waves:

Deposit 8 in. deep, velocity at about 3 in. from bottom found to average 0.9 ft. per second. Condition here is plainly one of deposit, and not of scour.

Station 4, upstream side of Pemberton bridge:

Upstream from this point the bottom and berms of canal are substantially scoured clean, but a short distance downstream from this point on the northerly edge of berm a deposit begins, and, going downstream, quickly spreads out to 5 ft. in width opposite to the penstocks of the Pemberton Mills, and below this gradually widens out, until at Union Street it covers the entire bed of the canal from north side over to foot of south slope.

At Pemberton Bridge, where entire bed is scoured clean, there is some irregularity found in the distribution of velocity, but the general average of a dozen or twenty observations ran about as follows:

	Feet per second
Mean velocity of entire cross-section	2.5
Velocity 3 in. from bottom at midchannel	1.6
At 10 ft. from north side	1.5
In corner next north wall (at deposit)	0.9

The observations at this point show that a velocity of 1.5 ft. per second prevents deposit or produces scour or a rolling along that keeps the bottom clean.

In general, these north canal observations show that the velocity necessary to prevent deposit or necessary to produce scour of grains of fine river silt and sand of sizes shown by following analysis (Table 22), and forming

TABLE 22.—MECHANICAL ANALYSIS OF AVERAGE SAMPLES OF SAND CAREFULLY COLLECTED FROM WITHIN 1/4 TO 1/2 IN. OF SURFACE AT ABOVE STATIONS; ANALYZED AT LAWRENCE EXPERIMENT STATION, MASSACHUSETTS STATE BOARD OF HEALTH

From J. R. Freeman's Report on Charles River Dam, 1903, p. 415

Number of sample	No. 1	No. 2	No. 3
Ten per cent finer than (diam. in millimeters)	0.12	0.15	0.04
Uniformity coefficient	1.40	1.70	3.60
Finer than 2.04 mm. (per cent by weight)	100.00	100.00	100.00
Finer than 0.93 mm. (per cent by weight)	99.60	99.60	100.00
Finer than 0.46 mm. (per cent by weight)	98.00	97.80	99.00
Finer than 0.316 mm. (per cent by weight)	95.50	93.40	97.80
Finer than 0.182 mm. (per cent by weight)	66.10	33.10	89.20
Finer than 0.105 mm. (per cent by weight)	4.30	0.90	32.40
Finer than 0.08 mm. (per cent by weight)	......	......	19.10
Finer than 0.04 mm. (per cent by weight)	......	......	9.60
Finer than 0.01 mm. (per cent by weight)	......	......	0.90

part of a mass deposited only less than 2 months before and not compacted by long standing, was not far from 1.3 to 1.5 ft. per second, this velocity being measured at a distance of from 3 to 6 in. from bottom.

These observations thoroughly disprove the oft-quoted, century-old, crude, unreliable observations of Dubuat.

The boiling and eddying of a current has much to do with its power to transport material in suspension. While this canal has riprap on its banks, its straightness and uniformity of section should offset any greater disturbance than is commonly found in natural streams, and should make the results of general applicability.

As an indication of the mean and maximum surface velocities corresponding to bottom velocities which were supposed to cause "a gradual destruction of the bed"—these latter being based upon the experiments of Dubuat, which Freeman characterizes as crude and unreliable—Ganguillet and Kutter have computed such values with the aid of

TABLE 23.—VELOCITIES BEYOND WHICH EROSION OF BED OF STREAM WILL TAKE PLACE, BASED UPON DUBUAT'S EXPERIMENTS[1]

Hering and Trautwine's Translation of Ganguillet and Kutter's "Flow of Water," p. 124

Column 2 gives the velocity at the bottom; column 3, the mean velocity as figured by Bazin's formula, $v = v_b + 10.9\sqrt{RS}$, in English measure, or an average value of $v = 1.31v_b$; column 4 contains the maximum surface velocity as figured by Bazin's formula, $v = v_{max} - 24.5\sqrt{RS}$ in English measure, or a mean value of $v = 0.83v_{max}$.

Nature of material forming bed	Bottom velocity, v_b ft. per sec.	Mean velocity, v ft. per sec. Bazin	Maximum surface vel., v_{max}, ft. per sec. Bazin
(1)	(2)	(3)	(4)
River mud, clay, specific gravity = 2.64	0.25	0.33	0.40
Sand, the size of anise seed, specific gravity = 2.55	0.35	0.46	0.55
Clay, loam, and fine sand	0.50	0.66	0.79
Sand, the size of peas, specific gravity = 2.55	0.60	0.79	0.95
Common river sand, specific gravity = 2.36	0.70	0.92	1.10
Sand, the size of beans, specific gravity = 2.55	1.07	1.40	1.69
Gravel	2.00	2.62	3.15
Round pebbles, 1-in. diam., specific gravity = 2.61	2.13	2.79	3.36
Coarse gravel, small cobblestones	3.00	3.93	4.73
Angular stones, flint, egg size, spec. gravity = 2.25	3.23	4.23	5.09
Angular broken stone	4.00	5.24	6.30
Soft slate, shingle	5.00	6.55	7.86
Stratified rock	6.00	7.86	9.43
Hard rock	10.00	13.12	15.75

[1] Freeman calls these experiments "crude and unreliable." Note that specific gravity seems to be of significance only when considered in connection with size; in other words, total weight of particles seems to be of greatest importance.

formulas of Bazin. These velocities, although probably untrustworthy,[1] are given in Table 23 as a rough indication of the relative velocities which were formerly supposed to result in some erosion. Such velocities would not necessarily have any relation to the velocities required to avoid deposition of solids in flowing streams.

Bazalgette found the following velocities in feet per second were necessary to move the bodies described: fine clay, 0.25; sand, 0.50; coarse sand, 0.66; fine gravel, 1.00; pebbles 1-in. diameter, 2.00; stones of egg size, 3.00.

Blackwell showed by experiments made for the British Metropolitan Drainage Commission that the specific gravity has a marked effect upon the velocities necessary to move bodies, as given in Table 24.

TABLE 24.—RELATION BETWEEN SPECIFIC GRAVITY AND VELOCITY OF WATER NECESSARY TO MOVE SUBSTANCES

Hering and Trautwine's Translation of Ganguillet and Kutter's "Flow of Water," p. 125

Nature of bodies	Specific gravity	Velocity in feet per second necessary to move bodies
Coal	1.26	1.25 to 1.50
Coal	1.33	1.50 to 1.75
Brickbat	2.00	1.75 to 2.00
Piece of chalk	2.05	
Oölite stone	2.17	2.00 to 2.25
Brickbat	2.12	
Piece of granite	2.66	
Brickbat	2.18	2.25 to 2.50
Piece of chalk	2.17	
Piece of flint	2.66	2.50 to 2.75
Piece of limestone	3.00	

Note that in both of the above quotations there is no discrimination among surface, mean, and bottom velocities.

The Metropolitan Sewerage Commission of New York, 1910, assumed the velocities given in Table 25 to be necessary to move solid particles.

[1] Ganguillet and Kutter say: "Whether and how far these velocities are reliable, we have not been able to determine."

TABLE 25.—CURRENTS NECESSARY TO MOVE SOLIDS
Metropolitan Sewerage Commission, New York

Kind of material	Velocity required to move on bottom	
	Feet per second	Miles per hour
Fine clay and silt	0.25	about 1/6
Fine sand	0.50	about 1/3
Pebbles half inch in diameter	1.0	about 2/3
Pebbles 1 in. in diameter	2.0	about 1 1/3

In general, it is found that a mean velocity of 1 ft. per second, or thereabouts, is sufficient to prevent serious deposition of organic sewage solids upon tidal flats, if the sewage is reasonably comminuted.[1]

The interesting experiments both of Professors Adeney and Letts of the Royal Commission, and Clark of the Massachusetts State Board of Health (the latter made in connection with Freeman's Report upon the Charles River Dam) conclusively point to the fact that the polluting organic matter is precipitated very much more rapidly in salt water than in fresh.[2] The danger of formation of sludge banks from the discharge of a given quantity of sewage into a body of salt water is greater therefore than in the case of a like body of fresh water. This is not a phenomenon depending upon the transporting power of flowing water, however, although it might be confused with it.

Self-cleaning Velocities in Sewers.—The transporting capacity of water is important on account of its bearing upon the possible clogging of sewers. The actual conditions of flow in the sewers must also be clearly borne in mind.

As is given in the diagrams showing the hydraulic elements of various sewer sections (Figs. 32 to 48), the velocity of flow in any sewer laid upon a given grade varies markedly with the depth of sewage flowing. Obviously, the quantity flowing also varies greatly, at different hours of the day, and on different days, as discussed in Chap. V. At times of low flow of sewage, the velocity may be so low that the stream will be able to transport only the finely comminuted suspended matter; the paper, street washings, and other foreign matter contained in the water will temporarily find lodgment upon the bottom and sides of the sewer. If the stranded matter is sufficient in amount, pooling of the sewage behind the obstruction will result until the pressure is sufficient to break through the obstruction and develop a velocity which will again pick up the arrested material and transport it. Owing to the grease

[1] Metropolitan Sewerage Commission of N. Y., *Rept.*, 1910; 434

[2] See Vol. iii, Second Ed., p. 268.

contained in the sewage, and because a velocity 30 to 40 per cent greater is required to pick up materials than to transport those already in suspension, the material will not be picked up again at the same velocity as that at which it was deposited, and obstructions may thus be formed and gradually built up to a point where sufficient velocity is developed to maintain a channel between the surface of the deposit and the crown of the sewer.

From the point of view of operation, it is important that the minimum velocities assumed in the design of the sewer, when flowing one-half full, two-thirds full,[1] or full, as the case may be, shall be adequate to keep it thoroughly flushed. It has been found that a mean velocity of 2½ ft. per second will ordinarily prevent deposits in combined sewers, and 2 ft. per second will ordinarily prevent deposits in separate sewers.

Mean velocities which are sufficient to prevent deposits are commonly called self-cleaning velocities. It is desirable that a mean velocity of 3 ft. per second, or more, shall be obtained where possible, and this limit should not be lowered in the case of inverted siphons under ordinary conditions. While lower minimum velocities have been used in some places, they have often been accompanied by more or less expense for removing sediment by which the sewers might in time become clogged. Slopes giving as low velocity as 1.5 ft. per second have been used in separate sewers, but they are undesirable and are likely to lead to greater cost in maintenance.

The main intercepting sewer at Columbus, Ohio, laid upon slopes of 0.61 ft. per 1,000 for the 30-in. section to 1.94 ft. per 1,000 for a 36-in. section, giving velocities from a minimum of 1.72 ft. to a maximum of 3.6 ft. per second (assuming the sewer to flow full and n to equal 0.015), has given considerable trouble from the collection of large quantities of sediment.

Part of the Boston Main Drainage Works consists of a tunnel 7.5 ft. in internal diameter and 7,166 ft. long, operated as an inverted siphon. The ordinary velocity through this tunnel at the inception of the works was about 1 ft. per second. To ascertain the extent of deposits under these conditions, water was pumped in at one end and the difference in level at the two ends was noted for the purpose of figuring the value of C in $v = C\sqrt{RS}$. It was assumed that when this value approximated 137 it would indicate that there were no deposits. The results of these experiments are given in Table 26. On the basis of this assumption, these figures indicate that deposits occurred with a velocity of approximately 1 ft. per second and did not occur with a velocity of approximately 4 ft. per second (Boston Main Drainage Report, 1885).

[1] It is customary to call the condition resulting in a two-thirds depth of flow, two-thirds full, irrespective of the relative quantities flowing.

TABLE 26.—EXPERIMENTS AT BOSTON TO DETERMINE VELOCITIES AT WHICH DEPOSITS OCCUR

Number of experiment	Mean velocity, feet per second	Value of C in $v = C\sqrt{RS}$	Liquid flowing
1	0.929	79.95	Sewage
2	0.998	82.00	Sewage
3	3.988	129.05	20 to 25 per cent sewage, 75 to 80 per cent salt water
4	0.965	109.66	Sewage
5	3.929	120.67	20 to 25 per cent sewage, 75 to 80 per cent salt water
6	3.897	146.31	Ditto
7	4.062	146.64	Ditto

TABLE 27.—OBSERVATIONS AT WORCESTER OF VELOCITIES AT WHICH DEPOSITS DO AND DO NOT OCCUR

Street	Kind of sewer	Size, inches	Approximate mean velocity, feet per second	Shape	Remarks upon deposit
Pink......	Storm	24 by 36	2.06	Egg	Deposit occurs
Pink......	Storm	18	1.47	Egg	Deposit occurs
Pink......	Storm	18	1.46	Egg	Deposit occurs
Pink......	Storm	18	1.17	Egg	Deposit occurs
Pink......	Storm	18	2.86	Egg	Deposit occurs
Pink......	Storm	18	1.13	Egg	Deposit occurs
Pink......	Storm	18	2.14	Egg	Deposit occurs
Highland...	Storm	18	3.74	Egg	No deposit
Highland...	Storm	18	3.02	Egg	No deposit
Highland...	Storm	12	2.26	Round	No deposit
North......	Combined	22 by 33	1.99	Egg	Deposit occurs
North......	Combined	18	1.94	Egg	No deposit
North......	Combined	18	2.25	Egg	No deposit
North......	Combined	18	1.72	Egg	No deposit
North......	Combined	18	2.61	Egg	No deposit
North......	Combined	15	2.56	Egg	No deposit
North......	Combined	15	1.54	Egg	No deposit
North......	Combined	12	4.63	Round	No deposit
North......	Combined	12	6.97	Round	No deposit

The Pink and Highland street sewers form a single line, beginning with a 12-in. round section. The figures begin at the bottom and should be read upward. There was no trouble until the velocity dropped to 2.14 ft. per second. The reason that trouble is experienced where the velocity should theoretically be 2.86 ft. per second probably lies in the flat grades on each side of it. In the case of the North street sewer, no trouble is experienced until the lower end is reached, where for about 450 ft. the velocity falls to 1.99 ft. per second. This is not so low as the velocities at several other places, but each of the latter is preceded by at least one section which has a good velocity.

H. P. Eddy,[1] gives his observations upon certain sewers in Worcester, Mass., in Table 27.

The Metropolitan Sewerage Commission of New York, in its sixth Preliminary Report, 1913, fixed from 2 to 5 ft. per second as a suitable range of velocities to prevent deposit from screened sewage from which the grit had first been removed, in a proposed siphon 2,300 ft. long and from 8 to 9 ft. in diameter, to be laid 110 ft. below the surface of mean low water to carry the sewage (99,000,000 gal. a day in 1915) from Manhattan Island to Brooklyn beneath the lower East River.

Minimum Allowable Grades.—The following opinions as to safe practice in selecting minimum grades were furnished, in 1913, to the authors by the engineers whose names are given.

In general, the minimum grades given in Table 28 for small separate sewers have been found safe, though steeper grades are always desirable. These grades are the least ordinarily permitted by the New Jersey State Board of Health. In its 1913 regulations governing the submission of designs, it stated:

> The sewers should have a capacity, when flowing half full, sufficient to carry twice the future average flow 25 years hence, plus a sufficient allowance for ground-water infiltration. When grades lower than those given are used, an explanation and reasons for the use of such grades should be included in the engineer's report.

TABLE 28.—MINIMUM GRADES IN SEPARATE SEWERS FOR 2-FT. VELOCITIES[1]

Diameter, inches	Minimum slope in per cent
4	1.2
6	0.6
8	0.4
10	0.29
12	0.22
15	0.15
18	0.12
20	0.10
24	0.08

[1] $n = 0.013$

James N. Hazlehurst stated that his practice had been largely in connection with sewer systems in the southeastern coast states, where there is much silt and running sand. Minimum grades were absolutely necessary to accomplish anything and he generally used grades lower than those recommended in textbooks. The minimum grade for each size of pipe sewer, which he ordinarily permitted, was: 6-in. sewer,

[1] *Jour. Assoc. Eng. Soc.*, 1904; **33**, 235.

0.33 per cent; 8-in., 0.25; 10-in. 0.20; 12-in., 0.17; 15-in., 0.15; 18-in., 0.12; 20-in., 0.10; 24-in., 0.08. When sewers were properly constructed he reported that he knew of no trouble from deposits when the grades were not lower than those stated. In Waycross, Ga., there were 8-in. pipe sewers on grades as flat as 0.24 per cent, which operated without giving trouble; on a few grades which were as flat as 0.10 per cent, however, the sewers were clogged from time to time and had to be rodded out.

Charles B. Burdick stated that it was the practice of Alvord and Burdick to secure grades that would give a velocity of 2 ft. per second in separate sewers flowing full or half full, and to reduce this to 1½ ft. per second, if necessary. Even on such grades they used flush tanks at the summits of the laterals, and if these velocities could not be obtained, special flush tanks were usually installed. On combined sewers they endeavored to secure 3 ft. velocity, but reduced it to 2 ft. if necessary. He stated:

> It is our practice to get all the grade we can at reasonable expense, and if it is impossible through physical conditions or cost to get the grade desired, we usually instal some means for flushing, with the idea of removing deposits. We have in several cases installed a specially capacious flush tank at the head of a main where an unusually flat grade is used, these especially flat grades coming more commonly on mains than laterals.

George G. Earl stated that the standard minimum grades for sewers in New Orleans, given in Table 29, were occasionally disregarded, because it had been necessary in some cases to lay considerable 8-in. pipe on grades as low as 0.25 per cent. The aim is to have a velocity of 2 ft. per second in a half-full 8-in. pipe, and a slightly increasing velocity in half-full sewers as the size increases. The sewers were of vitrified pipe up to 30-in. diameter, and either brick or concrete in larger sizes. Some of those over 30 in. in size are semielliptical in section, but on account of constant infiltration the volume of flow is sufficient at all times to render circular sections fairly satisfactory.

Better bottom grades are usually obtained in the drainage system at New Orleans, than in the sewers. The main drains have a V-shaped bottom, with transverse slopes of about 1:4; they are 4 to 25 ft. wide, with good bottom gradients which give velocities of 5 to 10 ft. per second when running full. The laterals enter them with inverts flush with the bottom at the sidewalls, and thus have the maximum grade practicable. Earl stated that the drainage system, particularly the vitrified pipe laterals from 10 to 30 in. in diameter, receive street washings and sweepings in dry weather when the flow is inadequate to remove them, and, consequently, a good deal of flushing and cleaning is required on account of these dry-weather accumulations.

TABLE 29.—MINIMUM GRADES ON NEW ORLEANS SEWERS

Diameter, inches	Slope, per cent	Diameter, inches	Slope, per cent	Diameter, inches	Slope, per cent
8	0.33	27	0.100	48	0.062
10	0.25	30	0.091	51	0.059
12	0.21	33	0.083	54	0.056
15	0.167	36	0.083	57	0.053
18	0.133	39	0.077	60	0.050
21	0.114	42	0.071	63	0.050
24	0.100	45	0.067	66	0.050

George W. Fuller stated that his drafting-room practice for separate pipe sewers was based on a 2-ft. velocity when half full, with a coefficient of roughness, n, of 0.013. This coefficient is also used for concrete sewers 24 in. in diameter and over, and 0.015 is used for brick sewers. Rather than go to the expense of pumping where the grades tend to make it necessary, the slopes giving the velocities mentioned are sometimes flattened. This is done, however, only after a careful examination of local conditions on the ground, and is not normal office practice. For instance, at Vincennes, Ind., in a sewerage system designed about 1910, Fuller made use of grades of 0.3 per cent with 8-in. pipe and in some cases a grade of only 0.25 per cent was used. J. R. McClintock reported subsequently for Fuller that an examination of the Englewood, N. J., sewerage system revealed a number of sewers with very low grades, which were apparently quite satisfactory. Six-inch sewers were discharging freely with grades as low as 0.35 per cent, and there were cases of 12-in. pipe with a grade of about 0.10 per cent, and 8-in. pipe with grades of 0.10, 0.15, and 0.20 per cent in satisfactory operation. There were other sections of this same system, however, where sewers with grades apparently no lower were partly clogged.

Fuller stated that in the case of separate sewers he was of the opinion that the depositing velocities would not have appreciable significance if substantially every day there were periods when the velocity approached 28 in. per second or more, and that he stated carefully to clients that where the slopes of sewers, more than two or three blocks removed from flush tanks at the head of a line, showed a velocity of less than 20 in. per second, care should be taken to flush the sewers either by a hose or some equivalent. In the case of combined sewers, he endeavored to secure a nominal minimum velocity of 2½ ft. per second. In practically every case where he has had occasion to study in detail the condition of such sewers, a heavy grit has been found deposited in them. If these deposits were not removed, they apparently decomposed and became more or less cemented by ferrous sulphide. The

result was that scouring velocity applicable to ordinary street wash would no longer suffice. This he found quite marked in Elizabeth, N. J., although the data are too meager to find place in a record of accurate information.

C. E. Grunsky stated that the minimum grades in Californian cities, reported to him by the engineers of the places named, were as given in Table 30. The city engineer of Stockton said that the grades in that city

TABLE 30.—MINIMUM GRADES IN CALIFORNIA CITIES, PER CENT

Size, inches	Stockton	Fresno	Modesto	Visalia	Sacramento
6	0.2	0.15	0.16	0.3	0.25
8	0.143	0.1	0.16	0.24	0.2
10	0.139	0.1	0.2	0.143	0.16
12	0.1	0.1	0.152	0.143	0.12
15	.....	....	0.09		
18	.....	....	.....	0.1	

have caused no trouble during the 25 years the sewers had been in service; these sewers carry only sewage, rain water being excluded. Once in a great while they have had some trouble from deposits at Fresno, due to sluggish flow, according to the city engineer. The city engineer of Visalia stated that he had made float measurements in the sewers and found that the actual minimum velocity when they were running one-third to one-half full, was 1.1 ft. per second in 10-in. sewers, and a velocity of 1.75 ft. per second was observed in an 18-in. sewer half full. The light grades caused no trouble in that city. The city engineer of Sacramento stated that the depth of flow in the sewers of his city did not average one-fourth of their diameters; in no case had there been any offensive deposits.

T. Chalkley Hatton in experiments with two 24-in. sewers discharging creek water carrying considerable clay, the grade being 0.077 per cent, found no appreciable sediment with the following depths in inches and velocities in feet per second:

Depth..................	5	12	12
Velocity.................	1.21	2.35	1.70

Alexander Potter stated that his general practice was to lay all sewers at grades giving a velocity, when half full, of at least 2 ft. per second and preferably 2½ ft. With grades giving velocities much less than 2 ft. per second when half full, flushing and frequent cleaning are necessary. In order to avoid pumping or costly construction, however, Potter has used very flat grades at times. At Harrison,

N. Y., about 5,000 ft. of 20-in. sewer were laid with a fall of only 0.11 per cent. As the average flow will never more than quarter fill the pipe, arrangement has been made to flush it automatically once a day. At Kingsville, Tex., in order to avoid pumping, sewers flushed automatically once a day have been laid on grades as low as 0.1 per cent for 18-in. and 15-in., 0.15 per cent for 12-in., 0.2 per cent for 10-in., and 0.33 per cent for 8-in. In the southern part of Texas where the land is very flat many 8-in. sewers have been laid with a fall of only 0.20 per cent. In Corpus Christi, Tex., Potter found that practically all 8-in. laterals had been laid with a minimum grade of 0.2 per cent, and were kept clean by frequent flushing.

The authors' practice (1928) is to endeavor to secure a velocity of at least 2.0 ft. per second in separate sewers and of 2.5 ft. per second in combined sewers, using $n = 0.015$ for pipe sewers 24 in. and smaller. There are instances, as noted in connection with the Worcester data, where lower velocities may be permitted if there is a high velocity immediately above or below. The actual velocity in the flatter length of sewer may then be higher than the computed velocity based upon the slope. All such cases would require special study before permitting a variation from the above limiting velocities.

Relation between Velocity and Depth of Flow in Sewers.—It is important to bear in mind that the velocity of flow in conduits in which the depth is slight is materially less than when full or nearly full. This

TABLE 31.—SLOPES REQUIRED TO PRODUCE A VELOCITY OF 2.0 FT. PER SECOND IN PIPE SEWERS, AT VARIOUS DEPTHS OF FLOW AND WITH VARIOUS ASSUMED VALUES OF n

Slope in feet per 100

Diameter of pipe, inches	Sewer full or half full				Sewer one-fourth full		
	$n =$ 0.013	$n =$ 0.014	$n =$ 0.015	$n =$ 0.016	$n =$ 0.015	$n =$ 0.016	$n =$ 0.017
4	1.25	1.48	1.84	2.17	4.88	5.68	7.10
6	0.67	0.81	0.98	1.14	2.37	2.82	3.41
8	0.44	0.53	0.62	0.75	1.46	1.71	2.02
10	0.29	0.35	0.42	0.50	1.02	1.17	1.37
12	0.21	0.26	0.31	0.37	0.76	0.88	1.04
15	0.16	0.19	0.23	0.27	0.53	0.62	0.74
18	0.12	0.14	0.17	0.19	0.39	0.45	0.53
21	0.10	0.11	0.14	0.16	0.31	0.35	0.43
24	0.08	0.10	0.11	0.13	0.25	0.29	0.34
27	0.07	0.08	0.09	0.11	0.21	0.24	0.29
30	0.06	0.07	0.08	0.09	0.18	0.20	0.23

consideration may be of particular importance with pipe sewers, in which, as pointed out in Chap. II, there are some experimental indications that the value of n increases as the depth of flow decreases.

For the larger sizes of sewers it is most convenient to compare the hydraulic elements for various depths of flow by means of diagrams such as Figs. 32 to 48. For pipe sewers, Table 31, showing the grades required to produce a velocity of 2.0 ft. per second for various depths of flow and with various assumptions as to the value of n, will be found useful.

HYDRAULIC ELEMENTS OF SOME STANDARD SEWER SECTIONS

In Figs. 32 to 48, inclusive, are given the hydraulic elements of certain standard sewer sections, which have been figured by the applica-

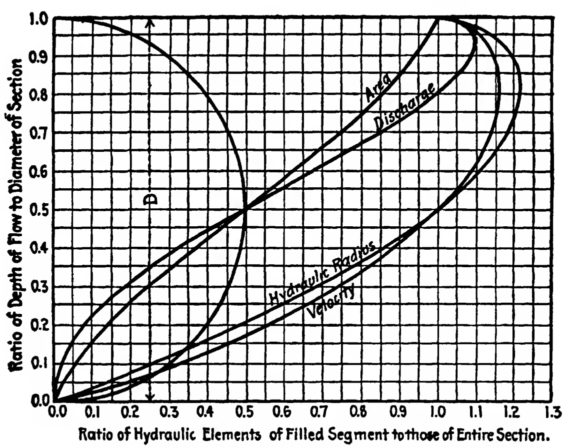


FIG. 32.—Hydraulic elements of small circular section by Kutter's formula. $n = 0.015$; $S = 0.005$; $D = 1$ ft.; area $= 0.785D^2$; wetted perimeter $= 3.1416D$; hydraulic radius $= 0.250D$.

tion of the principles outlined in Chap. II. The computation of the elements of sewer sections other than the circular is a rather long process and can be considerably lightened by using a planimeter where extreme accuracy is not required.

It must be remembered that, while the areas and hydraulic radii are computed by geometry, and depend only on the shape and size of the section, the other hydraulic elements shown are correct only for the particular sizes, slopes, and values of n, stated below the diagrams. They must, therefore, be used with caution, for conditions varying from these.

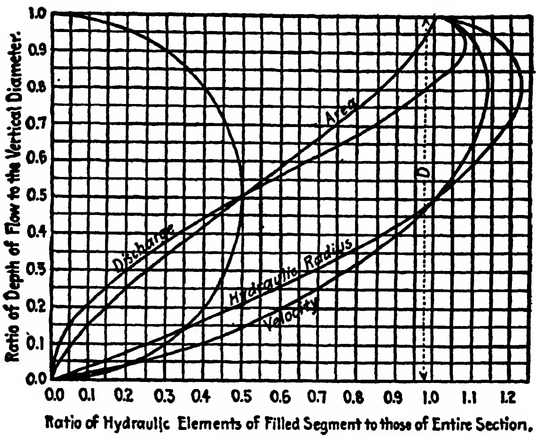


FIG. 33.—Hydraulic elements of large circular section by Kutter's formula. $n = 0.013$; $S = 0.0003$; $D = 7\frac{1}{2}$ ft.; area $= 0.785D^2$; wetted perimeter $= 3.1416D$; hydraulic radius $= 0.250D$.

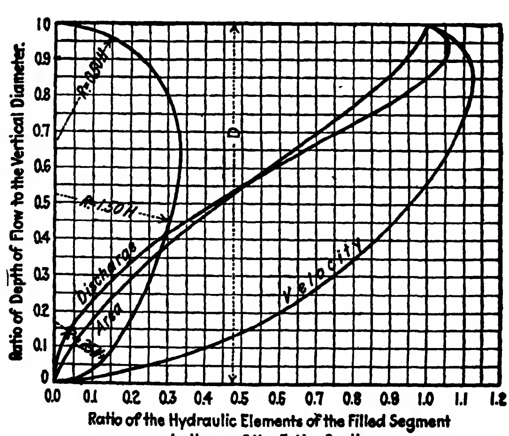


FIG. 34.—Hydraulic elements of egg-shaped section.
$n = 0.015$; $S = 0.000625$; $H = 4$ ft.; $D = 6$ ft. $= 1.254$ diam. equiv. circle; $H = 0.836$ diam. equiv. circle; area $= 1.1485H^2 = 0.5105D^2$; $R = 0.2897H = 0.1931D$.

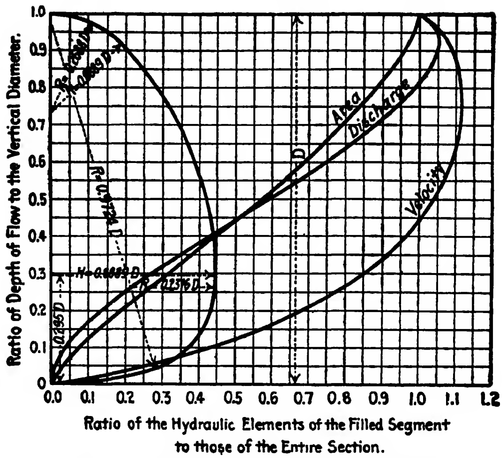


FIG. 35.—Hydraulic elements of catenary section.

n = 0.015; S = 0.000333; D = 7.44 ft.; vertical diameter = D = 1.063 diam. equiv. circle; area = $0.70277D^2$; R = $0.23172D$.

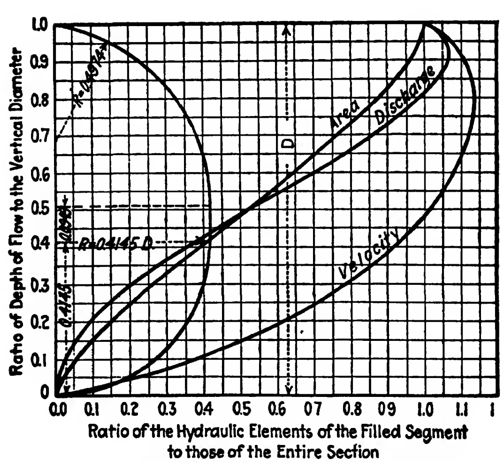


FIG. 36.—Hydraulic elements of gothic section.

n = 0.015; S = 0.000667; H = 3 ft.; horizontal diameter = H = 0.9167 diam. equiv. circle; vertical diameter = D = 1.1056 diam. equiv. circle; area = $0.9534H^2$ = $0.6554D^2$; R = $0.2737H$ = $0.2269D$.

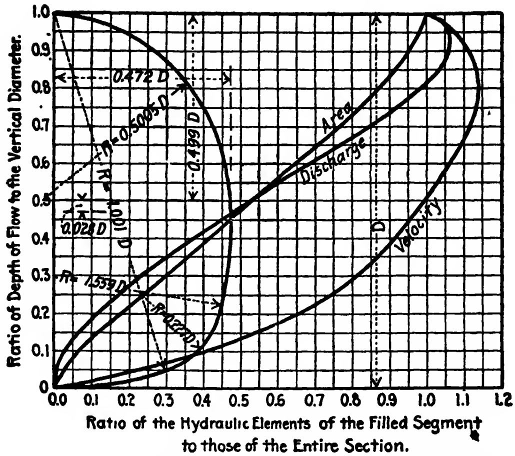


FIG. 37.—Hydraulic elements of basket-handle section.
$n = 0.015$; $S = 0.000333$; $D = 8$ ft. 10 in.; vertical diameter $= D = 0.999$ diam. equiv circle; area $= 0.7862D^2$; $R = 0.2464D$.

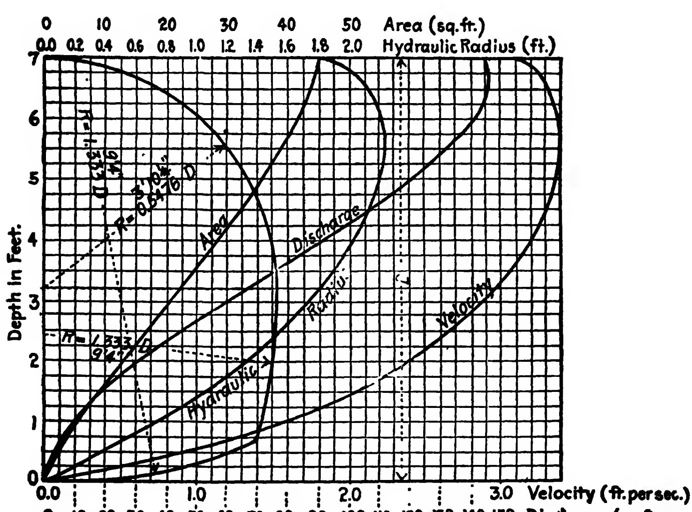


FIG. 38.—Hydraulic elements of horseshoe section, Wachusett type, by Kutter's formula.
$n = 0.013$; $S = 0.0003$.

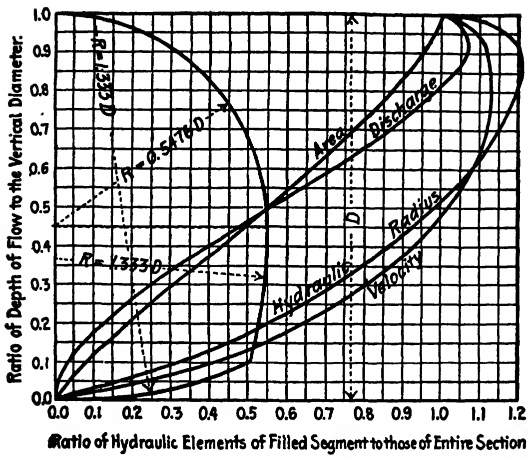


FIG. 39.—Hydraulic elements of horseshoe section, Wachusett type, by Kutter's formula.

$n = 0.013$; $S = 0.0003$; $D = 7$ ft.; horizontal diameter, $H = 7$ ft. 8 in.; area = 44.74 sq. ft. = $0.913D^2$; wetted perimeter = 24.26 ft. = $3.466D$; hydraulic radius = 1.841 ft. = $0.263D$.

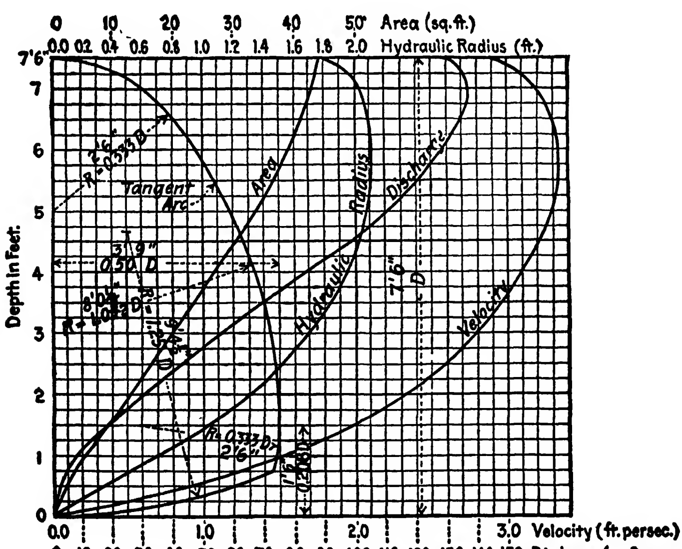


FIG. 40.—Hydraulic elements of special semielliptical section by Kutter's formula.

$n = 0.013$; $S = 0.0003$.

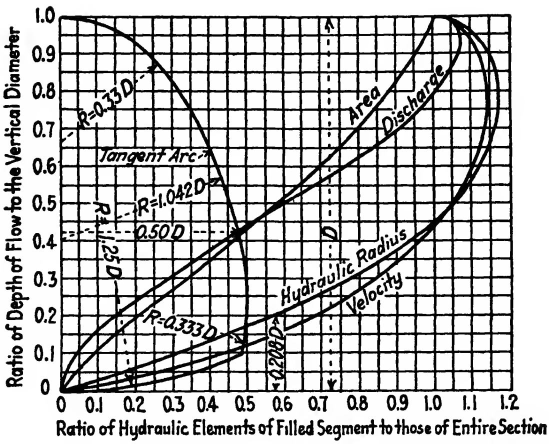


FIG. 41.—Hydraulic elements of Louisville semielliptic section by Kutter's formula.

$n = 0.013$; $S = 0.0003$; $D = 7\frac{1}{2}$ ft.; area $= 0.7831D^2$; wetted perimeter $= 3.26D$; hydraulic radius $= 0.242D$.

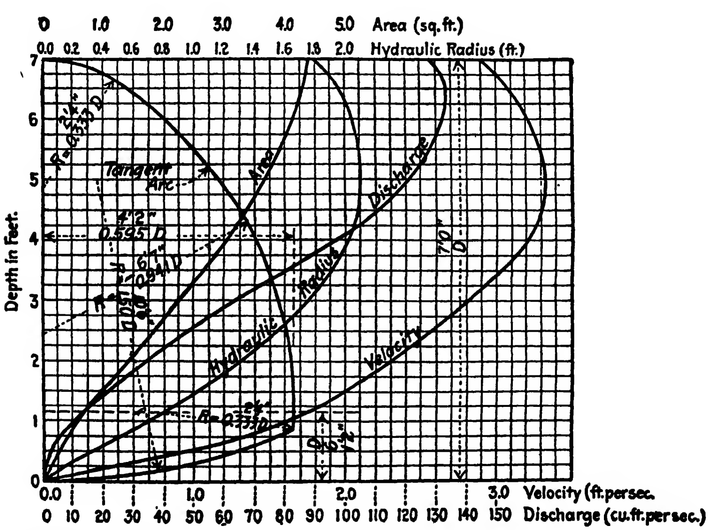


FIG. 42.—Hydraulic elements of semielliptic section by Kutter's formula.

$n = 0.013$; $S = 0.0003$.

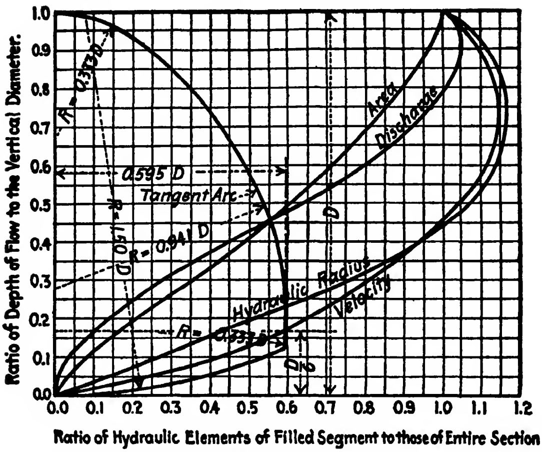


FIG. 43.—Hydraulic elements of special semielliptical section by Kutter's formula.

$n = 0.013$; $S = 0.0003$; $D = 7$ ft.; horizontal diameter, $H = 8$ ft. 4 in.; area $= 0.9D^2$ $= 44.1$ sq. ft.; wetted perimeter $= 3.508D = 24.56$ ft.; hydraulic radius $= 0.256D = 1.79$ ft.

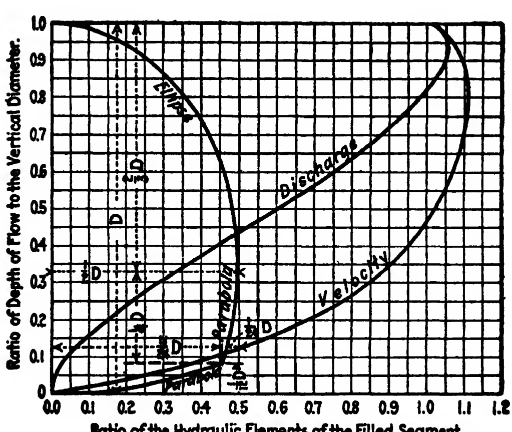


FIG. 44.—Hydraulic elements of Gregory's semielliptical section.

$n = 0.015$; $S = 0.0005$; $D = 10$ ft.; area $= 0.8176D^2$.

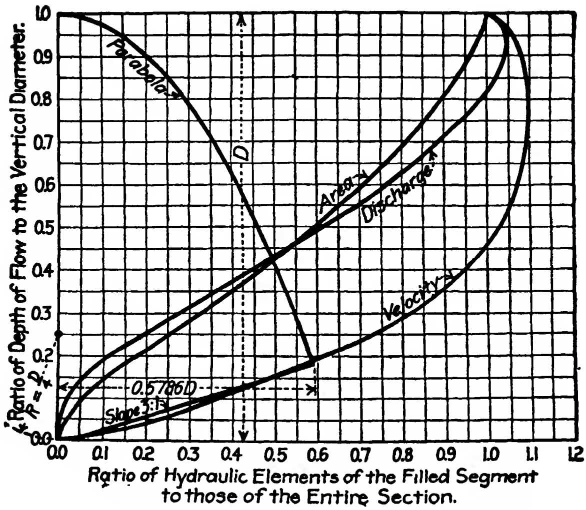


FIG. 45.—Hydraulic elements of parabolic section.
$n = 0.013$; $S = 0.001$; $D = 7$ ft. 4 in.; area $= 0.744D^2$; hydraulic radius $= 0.2245D$.

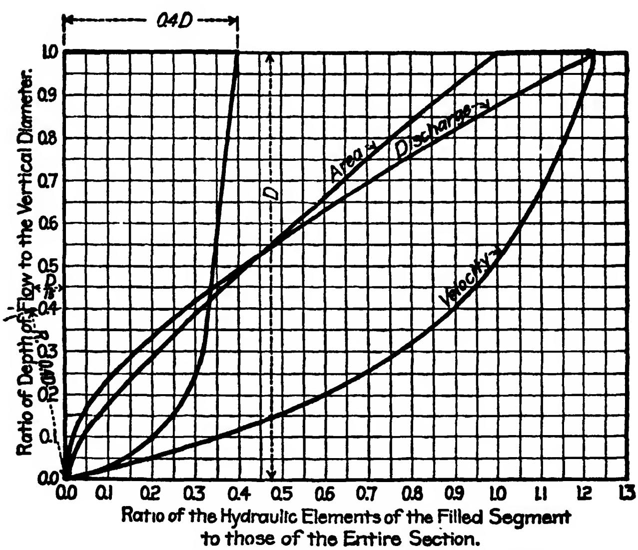


FIG. 46.—Hydraulic elements of U-shaped section.
$n = 0.013$; $S = 0.002$; $D = 2$ ft. 6 in.; area $= 0.6438D^2$; hydraulic radius $= 0.2047D$.

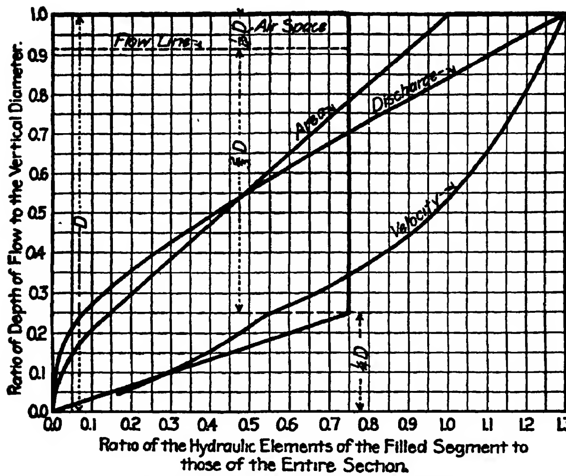


FIG. 47.—Hydraulic elements of rectangular section.
$n = 0.013$; $S = 0.001$; $D = 6$ ft.

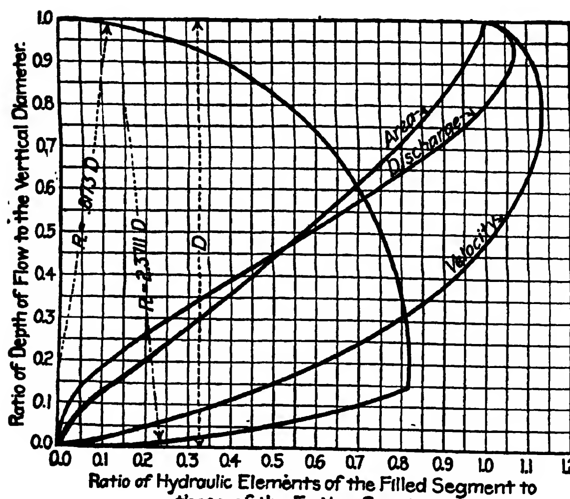


FIG. 48.—Hydraulic elements of semi circular section.
$n = 0.013$; $S = 0.001$; $D = 9$ ft. $2\frac{1}{8}$ in.; area $= 1.2697D^2$; hydraulic radius $= 0.2946\ D$.

No allowance has been made in any of these diagrams for change in the value of the coefficient n in the Kutter formula which may occur with change in the depth of flow, as previously noted in Chap. II.

Table 32 gives additional data regarding these same sections as well as others. Among other data, the table gives the wetted area and mean hydraulic radius of the filled sections in terms of the vertical diameter, and the relative value of the vertical diameter of the section in terms of the diameter of the equivalent circular section. By equivalent section is meant that section which has the same carrying capacity for a given slope and friction factor. The tables also give the actual size, slope, and friction factor upon which the table and curves were computed.

That there will be a slight difference in the hydraulic elements for various depths of flow, depending on the size of the section for which the diagram is computed, is shown by the first two lines of Table 32. The first line gives some of the hydraulic elements of a circular section based on a 12-in. pipe where $S = 0.005$ and $n = 0.015$. The second line was computed on the basis of a circular section 7 ft. 6 in. in diameter; $S = 0.0003$ and $n = 0.013$. The change in the value of n was made on account of the authors' practice which assumes 0.015 for pipe sewers and 0.013 for large concrete sewers. The slopes were also changed in order to approximate the slopes usually adopted in practice for the respective sizes.

Table 33 gives the principal hydraulic elements of the horseshoe section, Boston type, for various sizes of sewer.

As previously stated, that section is the best which for varying depths of flow maintains the hydraulic mean radius most nearly constant.

TABLE 32.—DATA RELATING TO HYDRAULIC ELEMENTS OF SEWER SECTIONS

Type of sewer	Depth of flow	Area of wet section in terms of vertical diam. D	Mean hyd. radius in terms of vert. diam. D	Proportionate depth at point of maximum		Proportionate value at point of maximum		Basis of exact comparison			Vertical diam. in terms of "d" diam. of equiv. circle	Reference to figure number
				Velocity	Discharge	Velocity	Discharge	Vertical diam. D in.	Slope S	Kutter's coef. n		
Circular	Full	$0.7854D^2$	$0.250D$	0.80	0.94	1.16	1.09	12.00	0.005	0.015	....	32
Circular	Full	$0.7854D^2$	$0.250D$	0.79	0.93	1.14	1.08	90.00	0.0003	0.013	....	33
Egg-shaped (standard)	Full[1]	$0.5105D^2$	$0.1931D$	0.85	0.94	1.13	1.06	72.00	0.000625	0.015	$1.254d$	34
Egg-shaped (standard)	⅔ Full[1]	$0.3359D^2$	$0.2105D$									
Egg-shaped (standard)	⅓ Full[1]	$0.1262D^2$	$0.1377D$									
Egg-shaped (new)	Full[2]	$0.4956D^2$	$0.1896D$	0.87	0.94	1.12	1.07	72.00	0.001–0.007	0.015		
Egg-shaped (new)	⅔ Full[2]	$0.3210D^2$	$0.2049D$									
Egg-shaped (new)	⅓ Full[2]	$0.1130D^2$	$0.1280D$									
Gothic	Full[3]	$0.6554D^2$	$0.2269D$	0.79	0.92	1.13	1.07	43.42	0.00067	0.015	$1.1056d$	36
Basket-handle	Full[3]	$0.7862D^2$	$0.2464D$	0.82	0.92	1.14	1.07	106.00	0.00033	0.015	$0.999d$	37
Catenary	Full[3]	$0.7028D^2$	$0.2317D$	0.77	0.93	1.11	1.06	89.28	0.00033	0.015	$1.063d$	35
Parabolic	Full	$0.7440D^2$	$0.2245D$	0.78	0.94	1.10	1.04	88.00	0.001	0.013	$1.0481d$	45
Semielliptical (Gregory's)	Full[4]	$0.8176D^2$	$0.2487D$	0.80	0.93	1.11	1.06	120.00	0.0005	0.015	....	44
Semielliptical (Louisville)	Full	$0.7831D^2$	$0.2404D$	0.77	0.93	1.14	1.07	90.00	0.0003	0.013	$1.000d$	41
Semielliptical (special)	Full	$0.900D^2$	$0.256D$	0.74	0.92	1.15	1.05	84.00	0.0003	0.013	$0.934d$	43
Five-centered (St. Louis)	Full[5]	$0.9747D^2 + 0.5683D - 0.1343$	....	....	....	.....	.....	....	....	0.013		
Horseshoe (Wachusett)	Full[3]	$0.913D^2$	$0.263D$	0.82	0.92	1.13	1.06	126.00	0.0004	0.015	$0.939d$	39
Horseshoe (Croton)	Full[3]	$0.850D^2$	$0.256D$	0.80	0.93	1.12	1.07	162.36	0.0004	0.015	$0.965d$	
Horseshoe (Boston)	Full	$0.8293D^2$	$0.2539D$	....	....	.....	.....	102.00	0.0004	0.015	$0.954d$	
Horseshoe (St. Louis)	Full[5]	$1.0139D^2$	$0.2750D$	....	....	.....	.....	....	....	0.013		
Semicircular	Full	$1.2697D^2$	$0.2946D$	0.80	0.92	1.14	1.07	110⅛	0.001	0.013	$0.8009d$	48
U-shaped	Full	$0.6438D^2$	$0.2047D$	Almost full depth		1.23	1.22	30.00	0.002	0.013	$1.1353d$	46
Rectangular	Full	$1.3125D^2$	$0.2865D$	Almost full depth		1.30	1.30	....	0.001	0.013	$0.7968d$	47
Rectangular	$1\frac{1}{2}D$	$1.1875D^2$	$0.4074D$	....	....	.....	.....	....	0.001	0.013	$0.7594d$	

[1] From "Hydraulic Tables" by P. J. Flynn. [2] From "Hydraulic Diagrams and Tables" by Garrett. [3] From "Hydraulic Diagrams" by Swan and Horton. [4] From Gregory, *Eng. News*, March 12, 1914. [5] From W. W. Horner, St. Louis, Mo.

NOTE.—Figures relating to velocity and discharge are correct only for the stated values of D, S, and n, but are approximately applicable to other values.

TABLE 33.—VALUES OF HYDRAULIC ELEMENTS OF HORSESHOE SEWER OF PROPORTION SHOWN IN FIG. 113c. COMPUTED BY BOSTON SEWER DEPARTMENT USING KUTTER'S FORMULA WITH $n = 0.013$ AND $S = 0.001$

Diameter		Area, a, square feet	Hydraulic mean radius R, feet	Wetted perimeter p, feet	$\frac{Q}{\sqrt{S}} = aC\sqrt{R}$,
Ft.	In.				
3	0	7.464	0.762	9.801	720.1
3	1	7.867	0.782	10.062	773.3
3	2	8.334	0.805	10.357	835.7
3	3	8.759	0.825	10.618	893.6
3	4	9.196	0.845	10.879	953.5
3	5	9.670	0.865	11.173	1,019.5
3	6	10.159	0.889	11.434	1,091.0
3	7	10.632	0.909	11 695	1,158.9
3	8	11 171	0.933	11.989	1,239.9
3	9	11.662	0.952	12.251	1,313.3
3	10	12.166	0.973	12.492	1,380.7
3	11	12.746	0.995	12.807	1,479.1
4	0	13.269	1 015	13.068	1,561.7
4	1	13 808	1 043	13.329	1,653.7
4	2	14.422	1.059	13.623	1,746.5
4	3	14.980	1 079	13.884	1,837.2
4	4	15.549	1.099	14.145	1,931.0
4	5	16 205	1.122	14.439	2,040.4
4	6	16.794	1.142	14.701	2,141.0
4	7	17.399	1.163	14.962	2,243.2
4	8	18.087	1.186	15 256	2,361.4
4	9	18.712	1.206	15 518	2,474.1
4	10	19.348	1.226	15.778	2,585.1
4	11	20.077	1.249	16.073	2,715.8
5	0	20.733	1.269	16.335	2,837.3
5	1	21.404	1.290	16.595	2,959.3
5	2	22.167	1.313	16.889	3,101.6
5	3	22.858	1.333	17 151	3,231.8
5	4	23.560	1.353	17.412	3,362.0
5	5	24.365	1.376	17.706	3,518.1
5	6	25 087	1.396	17.968	3,658.5
5	7	25 824	1.417	18.229	3,801.4
5	8	26.662	1.439	18.523	3,964.0
5	9	27.420	1.460	18.785	4,118.2
5	10	28.188	1 480	19.045	4,270.7
5	11	29 068	1.503	19.339	4,447.3
6	0	29.856	1.523	19.602	4,612.0
6	1	30.655	1.543	19.862	4,774.5
6	2	31.563	1.556	20.156	4,951.5
6	3	32.392	1.587	20.418	5,138.5
6	4	33.231	1.607	20.679	5,313.5
6	5	34.184	1.630	20.973	5,517.6

Table 33.—Values of Hydraulic Elements of Horseshoe Sewer of Proportion Shown in Fig. 113c. Computed by Boston Sewer Department Using Kutter's Formula with $n = 0.013$ and $S = 0.001$.—(*Continued*)

Diameter, Ft.	Diameter, In.	Area a, square feet	Hydraulic mean radius R, feet	Wetted perimeter p, feet	$\frac{Q}{\sqrt{S}} = aC\sqrt{R}$,
6	6	35 038	1.650	21.234	5,699.9
6	7	35.908	1.671	21.495	5,879.1
6	8	36.896	1.693	21.790	6,105.7
6	9	37.783	1.713	22.051	6,301.1
6	10	38.687	1.734	22.310	6,501.3
6	11	39.715	1.757	22.608	6,730.4
7	0	40 636	1.777	22.868	6,944.3
7	1	41.573	1.798	23.129	7,158.2
7	2	42.634	1.820	23.423	7,402.0
7	3	43.588	1.840	23.684	7,621.6
7	4	44.474	1.857	23.946	7,846.1
7	5	45.656	1.884	24.240	8,099.7
7	6	46.648	1.904	24.501	8,330.4
7	7	47.648	1.926	24,762	8,576.8
7	8	48.779	1.947	25.056	8,837.6
7	9	49.810	1.967	25.318	9,078.2
7	10	50.844	1.988	25.589	9,334.6
7	11	52.019	2.011	25.873	9,619.1
8	0	53.075	2.031	26.134	9,892.6
8	3	56.444	2.094	26.951	10,723.9
8	6	59.917	2.158	27.768	11,601.5
8	9	63.491	2.221	28.585	12,534.5
9	0	67.173	2.285	29.401	13,534.5
9	3	70.955	2.349	30.218	14,500.0
9	6	74.844	2.412	31.035	15,552.7
9	9	78.833	2.475	31.851	16,665.5
10	0	82.930	2.536	32.668	17,796.9
10	3	87.126	2.602	33.485	19,027.0
10	6	91.430	2.666	34.301	20,248.3
10	9	95.834	2.729	35.118	21,562.0
11	0	100.345	2.793	35.935	22,903.5
11	3	104.956	2.856	36.752	24,267.0
11	6	109.675	2.920	37.568	25,718.5
11	9	114.493	2.983	38.385	27,226.6
12	0	119.419	3.046	39.202	28,757.3
12	3	124.447	3.110	40.018	30,386.7
12	6	129.578	3.174	40.835	32,032.0
12	9	134.811	3.237	41.652	33,717.0
12	0	140.152	3.301	42.468	35,479.5

CONDITIONS OF FLOW ASSUMED IN DESIGN

In determining the proper size of sewers, it is first necessary to assume the condition of flow which is to exist in the sewers at time of maximum runoff. There is no uniformity of practice at the present time. It is the authors' practice to design sewers of all sizes on the assumption that they will flow full at times of maximum runoff. Some engineers assume that large sewers will be filled to the point of maximum discharge at times of maximum flow, and some assume that pipe sewers, at least in the smaller sizes, should carry the maximum estimated quantity when flowing half full.

There must always be some uncertainty as to the accuracy of estimates of quantity of sewage or storm water upon which the designs are based. Whatever provision should be made for unusual increments of flow or for factor of safety may be accomplished by suitable increases in the quantities assumed for design, proportioning the sewers to carry these amounts when full. This method permits the direct application of judgment and the results of experience, and would seem more likely to result in proper design than the making of a blanket allowance, as in designing sewers to flow half full.

Examination of Sewer Design with Reference to Minimum Flow Conditions.[1]—Economic considerations generally require the construction of main or intercepting sewers to meet future rather than present needs. The length of the period to be considered will be determined by the attendant circumstances, but in general such sewers are designed to meet the needs of a period of from 30 to 50 years in the future. As a result of this, the flow in the sewer for a long period of time will be much below the normal for which it is designed.

It is necessary, therefore, after designing a sewer for a given service in the future, to consider the actual conditions of operation likely to arise under dry-weather or minimum flow during the first few years after its construction, in order to make certain that the velocities will not be so low, for significant periods of time, as to cause serious deposits in the sewer, the removal of which would involve unwarranted cost. The construction of a sewer to serve for the long periods assumed above would be unwarranted if the cost thus resulting should exceed the cost of building a smaller sewer in the first instance, to serve for a shorter period of time and until the anticipated growth had developed in some degree, and then building a second sewer to take care of the additional sewage flow resulting from the added growth. While the latter plan would involve greater cost of construction, enough might be saved in fixed charges and in the cost of operation, in the early years of the use of the sewer, to more than cover this increased cost.

[1] See, also, Chap. VI on Separate Sewers, and Chap. IX on Storm Drains and Combined Sewers.

TABLE 34.—SIZE OF PROPOSED ACUSHNET INTERCEPTING SEWER, NEW BEDFORD, MASS., BASED UPON METCALF AND EDDY'S ESTIMATE OF REQUIRED CAPACITIES

Sewer at	Station or distance from point of discharge, feet	Length of interval, feet	Population tributary in 1910	Population tributary in 1940	Resulting Q in m.g.d.: Dry weather 125 g.c.d. = m.g.d. 1910	Dry weather 1940	Storm flow 400 g.c.d. = m.g.d. 1910	Storm flow 1940	Sewer: Dia. inches	Slope per 1,000 ft.	Fall in feet	Flowing full (1940): Capacity c.f.s.	Flowing full (1940): Velocity feet per second	Dry-weather flow (1910): c.f.s.	Depth in feet	Velocity, feet per second	Assumed elevation of invert
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)
North End	37,300	......	242	......	0.03	....	0.1	....	..	...	....	.....	...	....	....	....	10.68
Nash Road	36,000	1,300	5,072	15,000	0.63	1.9	2.0	6.0	36	0.7	0.91	(17.5)	2.5	....	....	....	9.77
																	9.27
Coffin	33,800	2,200	10,308	28,120	1.29	3.5	4.1	11.2	42	0.5	1.10	9.3	2.2	1.0	0.5	1.1	8.17
Avenue												(22.5)	(2.4)				7.67
Sawyer	32,200	1,600	21,284	65,240	2.66	8.2	8.5	26.1	48	0.50	0.80	27.2	2.9	2.0	0.6	1.4	6.87
Street													(2.6)				6.37
Acushnet	29,100	3,100	30,780	76,800	3.85	9.6	12.3	30.7	54	0.50	1.55	40.4	3.1	4.1	1.0	1.7	4.82
Avenue												(45)	(2.8)				4.32
Willis	25,400	3,700	49,056	110,980	6.14	13.9	19.6	44.4	60	0.40	1.48	47.5	3.0	6.0	1.2	1.7	2.84
Street												(52)	(2.7)				2.34
Elm Street	22,300	3,100	54,976	123,060	6.87	15.4	22.0	49.2	66	0.40	1.24	68.6	2.9	9.5	1.4	2.0	+6.10
																	+0.60
Howland	18,600	3,700	73,504	157,140	9.19	19.6	29.4	62.9	72	0.33	1.22	76.1	2.7	10.6	1.5	1.9	−0.62
Street																	−1.12
Rivet	16,400	2,200	98,252	202,140	12.3	25.2	39.3	80.8	78	0.33	0.73	97.2	2.9	14.3	1.69	2.0	−1.85
Street																	−2.35
Cove Road	13,600	2,800	105,500	220,695	13.2	27.6	42.2	88.3	{90[1] 84[2]	0.30	0.84	132.	3.0	19.0	1.91	2.1	−3.19
Screening Station	7,900	5,700	....	....	....	....	....	....	{90[1] 84[2]	0.30	1.71	132. (85.5 m.g d.)	3.0+	20.8	....	....	−4.9
Shore Point	3,200	4,700	....	....	....	....	....	....	{90[1] 84	....	....						
Harbor Discharge	0	3,200															
Total....	.....	37,300															

[1] Horseshoe type. [2] Circular type. g.c.d. = gallons per capita daily: m.g.d. = million gallons daily.
Note.—Amounts in parentheses indicate actual amounts corresponding to selected sizes of sewers, where other figures denote required figures.

An example of such a computation is shown in Table 34. It will be noted in column 14 that the estimated velocities under full flow, 30 years after the construction of the sewer, range from 2.4 to 3.1 ft. per second, whereas the velocities for the anticipated dry-weather flow at the beginning of the period range, in general, from 1.7 ft. to 2.1 ft. per second, though at the head of the sewer, velocities as low as 1.1 ft. per second were anticipated.

In such a consideration of cost proper weight should be given to the inconveniences to the public resulting from the construction of the second or relief sewer, to the changes in real estate development likely to take place and to affect the cost and practicability of obtaining rights of way, and to any unfavorable conditions of flow which may be caused by carrying portions of the sewage in two sewers instead of concentrating the flow in one sewer.

Where velocities are to be moderate or low, it is desirable that the sewer sections and slopes should be so designed that the velocity of flow will increase progressively, or at least that appropriate velocities will be maintained, in passing from the inlets to the outlet of the sewer, so that solids washed into the sewer and picked up and transported by the flowing stream may be carried through and out of the sewer, and not be dropped at some point owing to a decrease in velocity.

It is obvious that the velocity of flow is but one of many factors involved which must be given consideration in such an economic study; nevertheless, it is one which should be carefully weighed and not lost sight of.

Velocity in Submerged Sewers.—Computations relative to the dry-weather and minimum flows in submerged sewers, particularly such as sewer outfalls, must also be made, for here the conditions tending toward clogging of the sewer are particularly aggravated. Unless grit chambers or other devices for removing the heavy mineral matter are provided, the danger of clogging may be serious. This danger arises from the fact that where the pipe is submerged, flow takes place in the entire cross-section, and with a given rate of flow the velocity may thus be reduced to exceedingly small limits.

Fortunately, however, the matter in suspension, if of organic character only, tends to remain in a semiflocculent condition, buoyed up in part on account of its low specific gravity and in part by the gas formed by its putrefaction, so that if the sewer does discharge under substantial velocity from time to time during the day, or even at longer intervals, the flow may maintain the sewer reasonably free from clogging deposits.

If such outfalls are into salt water, the effect of the difference in specific gravity of the two liquids is to be borne in mind, as well as the fact

that in salt water the suspended organic matter is precipitated more quickly than in fresh water.[1]

Flush Tanks for Dead Ends.—The difficulty of obtaining adequate velocities of flow in the extremities of the sewer pipe system, where the grades are very flat, is met by the use of flush tanks or by flushing the sewers periodically in other ways. Such devices, though necessary under certain conditions, are at best a source of annoyance and expense on account of the difficulty of making them operate at regular intervals automatically and of the expense of furnishing water for the purpose of flushing. Moreover, the action produced in the sewer by the discharge from the flush tank is a purely local one, as the influence of the flood wave is felt for but a short time and to a comparatively short distance, as explained in Chap. XVI.

MAXIMUM VELOCITIES

Eroding Effect of High Velocities.—Most of the evidence available indicates that the erosive effect of clean water flowing at high velocities is so slight as to be negligible. F. C. Scobey, in *Bull.* 852, U. S. Bureau of Agriculture, gives a photograph showing that after 8 years the paint marks on the interior of the concrete lining of the Niagara Falls power tunnel were still clear, although subjected to water flowing with a velocity of 28 ft. per second. On the other hand, the second annual report of the (Massachusetts) Metropolitan Water and Sewerage Board cites a case in which granite and cast iron in the bottom of a gate house at one of the dams had been badly eroded in 8 years. It seems probable, however, that this action resulted mainly from the effect of a vertical drop rather than from the high velocity of the flowing water.

Erosion of Sewer Inverts.[2]—The erosive effect of sewage upon sewer inverts of different kinds would be unimportant in the case of the separate system were it not for the fact that sand, gravel, or other silicious material does find its way into many such sewers. In the combined system, which has to deal with larger quantities of silicious material as well as with rain water and sewage, the effect may be more important. The rapidity of the erosive action will depend not only upon the velocity of flow, but also upon the character of the material transported, arenaceous material, on account of its greater hardness, being much more destructive in its influence than argillaceous or limestone. Vitrified sewer pipe is resistant to erosion and has been laid successfully upon very steep grades. In large combined sewers, it has generally been customary to line concrete or brick sewers, in the invert at least, with vitrified brick, where the velocity of flow is in excess of 8 ft. per second, although some engineers have used as low a

[1] Compare Vol. iii, p. 268.

[2] See also discussion on "Wear of Sewer Inverts" on p. 452.

limit as 4 ft. per second. Wrought-iron or steel inverts have been used in some very steep sewer outfalls; in others, a depressed channel has been made in the main sewer, lined with split tile, vitrified brick, or steel, large enough to carry the dry-weather flow, the remainder of the invert being formed in concrete or lined with vitrified brick so that in case of need of repairs, it should be possible to get into the sewer during the dry-weather season to make the repairs without interruption of service.

The Metropolitan Sewerage Commission of New York reported in 1910, with reference to erosion in the outlets of the sewers inspected, that few cases were found where the bricks of the inverts were actually worn away. In a few places in the upper west side of Manhattan, the upstream edges of the bricks were rounded off as a result of the high velocity of the sewage flow. In a large number of the sewers, the mortar was worn from the joints in the brickwork of the invert. In some places the mortar had been worn away only to a slight depth while at other places it had been cut out by the sewage to the full depth of the brick.

In combined sewers at St. Louis, with grades ranging from 0.2 to 2 per cent, averaging about 0.5 per cent for sewers more than 5 ft. in diameter, and about 1 per cent for those of smaller sections, vitrified-clay pipes were stated by E. A. Hermann[1] to show no appreciable wear after about 35 years use, vitrified-brick inverts to show no appreciable wear after about 12 years, and inverts of ordinary sewer brick to show some wear after about 3 years' service and from 2 to 4 in. wear after a use of 30 years.

The 14-in. cast-iron siphon at Norfolk, Va., noted on page 599 was worn through at two points after 39 years' service, a V-shaped groove having been worn in the invert.

For reference to repairs of worn invert see Vol. II, First Ed., p. 516.

[1] *Eng. News*, 1904; **51**, 120.

CHAPTER IV

MEASUREMENT OF FLOWING WATER

Methods of Measurement.—The flow in or discharge from sewers or drains may be measured more or less closely by the following methods, the choice depending upon the purpose of the measurement and the conditions found:

1. By weighing the discharge for a given period of time in tanks or other receptacles.
2. By measuring the discharge for a given period of time in tanks or other receptacles, the contents of which can be accurately gaged.
3. By orifice.
4. By weir.
5. By meter of the Venturi type.
6. By Venturi flume.
7. By current meter.
8. By floats, dyes, chemicals, etc.
9. By computation from observed depth of flow and slope, where a condition of steady, uniform flow exists.
10. By computation from observed depth of flow at a critical section, when a condition of steady nonuniform flow exists.

The use of the Pitot tube, which has proved so useful in water-pipe gagings, is impracticable in sewer gagings, on account of the suspended matter contained in the sewage. Water meters containing moving parts are also inapplicable for the same reason. Nozzles are generally of little use on account of lack of pressure.

In the following paragraphs will be found a brief discussion, with accompanying formulas, tables, and curves for convenience in computation, relating to measurement by orifice, weir, Venturi-type meter, Venturi flume, current meter, and floats, dyes, and chemicals.

The application of the formulas discussed in Chap. II to the estimation of the quantity of sewage flowing in a sewer requires no explanation, the method being at best an approximation dependent upon the steadiness of the flow at the time of observation and the precision with which the coefficient of roughness is assumed for the existing conditions. Nevertheless, this method (No. 9 above) is the one which has been most used in ordinary sewerage work, and is sufficient for the needs of the superintendent of sewers in his everyday practice. For special investi-

gations, one of the other methods suggested should generally be used. The method selected will depend upon the facilities at hand, the degree of precision required, and the conditions under which the sewer was built and is operating. When it is practicable to ensure a critical section at some point, and to keep a record of depths at that point, computations based thereon should give results which are free from some of the inaccuracies of method No. 9.[1]

For a further discussion upon the measurement of flow in sewers, reference may be had to Chap. X.

Discharge through Orifices.—In accordance with Torricelli's theorem that the velocity of flow through an orifice is equal to the velocity acquired by a freely falling body in a space corresponding to the head over the orifice, the discharge through an orifice is

$$Q = cAv = cA\sqrt{2gH}$$

One reason for the need of the coefficient c, is the fact that the cross-section of the jet, at a point a short distance outside the orifice, has generally a somewhat smaller area than that of the orifice itself, the reduction in area depending upon the character of the orifice. When the edge is sharp so that the water does not adhere to the orifice, the coefficient is at a minimum or the reduction in area is at a maximum. When, on the other hand, the orifice is shaped to a bell mouth, the coefficient is at a maximum and the cross-section of the jet may be nearly equal to that of the orifice itself.

The section at which this reduction in area is at a maximum is known as the "contracted vein" and experiment indicates that the velocity of the water follows Torricelli's law literally in this section. The section of the contracted vein generally lies at a distance from the orifice of five-tenths to eight-tenths of its least diameter.

Table 35, from Hughes and Safford's "Hydraulics," shows the approximate variation in coefficients of discharge for a circular orifice of diameter 0.033 ft. and for heads of from 1 to 10 ft.

[1] The committee of the Am. Soc. C. E. on Irrigation Hydraulics reported in January 1928 (*Proc. Am. Soc. C. E.*, March 1928, 179) that "the experimental work done at Ft. Collins during the past summer on the Critical Depth Flow Meter has not been conclusive. For some reason, as yet unexplained, the device appears to be more sensitive to submergence than was expected."

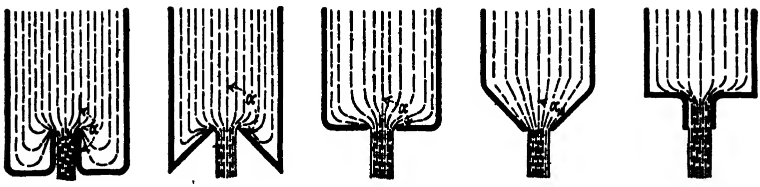


TABLE 35.—APPROXIMATE VARIATION IN COEFFICIENTS

	A		*B*		*C*	*D*					*E*
α	180°	157½°	135°	112½°	90°	67½°	45°	22½°	11¼°	5¾°	0°
c	0.541	0.546	0.577	0.606	0.632	0.684	0.753	0.882	0.924	0.949	0.966

The "standard orifice," as generally defined, is one in which the edge of the orifice which determines the jet is such that the jet upon leaving it does not again touch the wall of the orifice. Practically, this result is obtained by having the outside of the orifice bevelled and its throat cylindrical or prismatic in shape with an axial length between $\frac{1}{16}$ and $\frac{1}{8}$ in., depending upon the thickness of the plate. Merriman defines it as signifying that:

The opening is so arranged that the water in flowing from it (the orifice) touches only a line, as would be the case in a plate of no thickness. To secure this result, the inner edge of the opening has a square corner which alone is touched by the water . . . The orifice in a thin plate is often used to express the condition that the water shall only touch the edges of the opening along the line. This arrangement may be regarded as a kind of standard apparatus for the measurement of water.

Hughes and Safford,[1] have defined the standard orifice as follows:

If an orifice in a thin wall is set far enough from the side of the vessel or channel to secure full contraction of the jet, is round or square, and has no dimensions greater than 1 ft. (for which shapes and dimensions reliable coefficients are available), it is called a standard orifice.

Tables 36 and 37 contain coefficients of discharge for vertical standard orifices.

Weirs.—One of the most accurate methods of measuring water is by the use of a weir, provided the conditions under which the coefficients of discharge of given types of weirs were determined are approximately reproduced in the gagings.

The most common types of weirs are the rectangular, the triangular, and the trapezoidal weir.

[1] "Hydraulics," First Edition, 130.

Table 36.—Hamilton Smith, Jr.'s Coefficients of Discharge for Vertical Circular Orifices

Applicable strictly to vertical circular orifices in a flat, rigid, thin wall, with full contraction of the jet; and for the dimensions and heads given. Direct interpolation should be made for intermediate values.

Head on center of orifice, feet	Diameter of the orifice in feet												
	0.02	0.03	0.04	0.05	0.07	0.10	0.12	0.15	0.20	0.40	0.60	0.80	1.0
0.3	.....	.....	.....	0.637	0.628	0.621	0.612	0.607					
0.4	.....	.....	0.637	0.631	0.624	0.618	0.612	0.605					
0.5	.....	0.643	0.633	0.627	0.621	0.615	0.610	0.605	0.599	0.593	0.585		
0.6	0.655	0.640	0.630	0.624	0 618	0.613	0.609	0.605	0.600	0.594	0.588	0.581	
0.7	0.651	0.637	0.628	0.622	0.616	0.611	0.607	0.604	0.601	0.595	0.590	0.585	0.580
0.8	0.648	0.634	0.626	0.620	0.615	0.610	0.606	0.603	0.601	0.596	0.591	0.587	0.583
0.9	0.646	0.632	0.624	0.618	0.613	0.609	0.605	0.603	0.601	0.597	0.592	0.589	0.585
1.0	0.644	0.631	0.623	0.617	0.612	0.608	0.605	0.603	0.600	0.597	0.593	0.590	0.586
1.2	0.641	0.628	0.620	0.615	0.610	0.606	0.604	0.602	0.600	0.598	0.594	0.592	0.589
1.4	0.638	0.625	0.618	0.613	0.609	0.605	0.603	0.601	0.600	0.599	0.595	0.593	0.591
1.6	0.636	0.624	0.617	0.612	0.608	0.605	0.602	0.601	0.600	0.599	0.596	0.594	0.592
1.8	0.624	0.622	0.615	0.611	0.607	0.604	0.602	0.601	0.599	0.599	0.597	0.594	0.594
2.0	0.632	0.621	0.614	0.610	0.607	0.604	0.601	0.600	0.599	0.599	0.597	0.595	0.594
2.5	0.629	0.619	0.612	0.608	0.605	0.603	0.601	0.600	0.599	0.599	0.598	0.596	0.595
3.0	0.627	0.617	0.611	0.606	0.604	0.603	0.601	0.600	0.599	0.599	0.598	0.597	0.596
3.5	0.625	0.616	0.610	0.606	0.604	0.602	0.601	0.600	0.599	0.599	0.598	0.597	0.596
4.	0.623	0.614	0.609	0.605	0.603	0.602	0.600	0.599	0.599	0.598	0.597	0.597	0.596
5.	0.621	0.613	0.608	0.605	0.603	0.601	0.599	0.599	0.598	0.598	0.597	0.596	0.596
6.	0.618	0.611	0.607	0.604	0.602	0.600	0.599	0.599	0.598	0.598	0.597	0.596	0.596
7.	0.616	0.609	0.606	0.603	0.601	0.600	0.599	0.599	0.598	0.598	0.597	0.596	0.596
8.	0.614	0.608	0.605	0.603	0.601	0.600	0.599	0.598	0.598	0.597	0.596	0.596	0.596
9.	0.613	0.607	0.604	0.602	0.600	0.599	0.599	0.598	0.597	0.597	0.596	0.596	0.595
10.	0.611	0.606	0.603	0.601	0.599	0.598	0.598	0.597	0.597	0.597	0.596	0.596	0.595
20.	0.601	0.600	0.599	0.598	0.597	0.596	0.596	0.596	0.596	0.596	0.596	0.595	0.594
50. (?)	0.596	0.596	0.595	0.595	0.594	0.594	0.594	0.594	0.594	0.594	0.594	0.593	0.593
100. (?)	0.593	0.593	0.592	0.592	0.592	0.592	0.592	0.592	0.592	0.592	0.592	0.592	0.592

For heads of over 100 ft. use $c = 0.592$. (Hughes and Safford's "Hydraulics," First Edition, p. 144.)

TABLE 37.—SMITH'S COEFFICIENTS OF DISCHARGE FOR VERTICAL SQUARE ORIFICES

Applicable strictly only to vertical square orifices in a flat, rigid, thin wall, with full contraction of the jet and for dimensions and heads given. Direct interpolation should be made for intermediate values.

Head on center of orifice, feet	Side of square in feet												
	0.02	0.03	0.04	0.05	0.07	0.10	0.12	0.15	0.20	0.40	0 60	0.80	1.0
0.3	.....	.....	.....	0.642	0.632	0.623	0.616	0.610					
0.4	.....	.....	0.643	0.637	0.628	0.621	0.615	0.609					
0.5	.....	0.648	0.639	0.633	0.625	0.619	0.614	0.609	0.603	0.597	0.588		
0.6	0.660	0.645	0.636	0.630	0.623	0.617	0.613	0.610	0.604	0.598	0.592	0.580	
0.7	0.656	0.642	0.633	0.628	0.621	0.616	0.612	0.609	0.605	0.600	0.594	0.589	0.582
0.8	0.652	0.639	0.631	0.625	0.620	0.615	0.611	0.608	0.605	0.600	0.596	0.591	0.588
0.9	0.650	0.637	0.629	0.623	0.619	0.614	0.610	0.608	0.605	0.601	0.598	0.593	0.590
1.0	0.648	0.636	0.628	0.622	0.618	0.613	0.610	0.608	0.605	0.602	0.599	0.595	0.592
1.2	0.644	0.633	0.625	0.620	0.616	0.611	0.609	0.607	0.605	0.603	0.600	0.598	0.595
1.4	0.642	0.630	0.623	0.618	0.614	0.610	0.608	0.606	0.605	0.604	0.601	0.599	0.598
1.6	0.640	0.628	0.621	0.617	0.613	0.609	0.607	0.606	0.605	0.605	0.601	0.600	0.599
1.8	0.638	0.627	0.620	0.616	0.612	0.609	0.607	0.606	0.605	0.605	0.603	0.601	0.600
2.0	0.637	0.626	0.619	0.615	0.612	0.608	0.606	0.606	0.605	0.605	0.604	0.601	0.600
2.5	0.634	0.624	0.617	0.613	0.610	0.607	0.606	0.606	0.605	0.605	0.604	0.602	0.601
3.0	0.632	0.622	0.616	0.612	0.609	0.607	0.606	0.606	0.605	0.605	0.604	0.603	0.602
3.5	0.630	0.621	0.615	0.611	0.609	0.607	0.606	0.605	0.605	0.605	0.604	0.603	0.602
4.0	0.628	0.619	0.614	0.610	0.608	0.606	0.606	0.605	0.605	0.605	0.603	0.603	0.602
5.	0.626	0.617	0.613	0.610	0.607	0.606	0.605	0.605	0.604	0.604	0.603	0.602	0.602
6.	0.623	0.616	0.612	0.609	0.607	0.605	0.605	0.605	0.604	0.604	0.603	0.602	0.602
7.	0.621	0.615	0.611	0.608	0.607	0.605	0.605	0.604	0.604	0.604	0.603	0.602	0.602
8.	0.619	0.613	0.610	0.608	0.606	0.605	0.604	0.604	0.604	0.603	0.603	0.602	0.602
9.	0.618	0.612	0.609	0.607	0.606	0.604	0.604	0.604	0.603	0.603	0.602	0.602	0.601
10.	0.616	0.611	0.608	0.606	0.605	0.604	0.604	0.603	0.603	0.603	0.602	0.602	0.601
20.	0.606	0.605	0.604	0.603	0.602	0.602	0.602	0.602	0.602	0.601	0.601	0.601	0.600
50. (?)	0.602	0.601	0.601	0.601	0.601	0.600	0.600	0.600	0.600	0.600	0.599	0.599	0.599
100. (?)	0.599	0.598	0.598	0.598	0.598	0.598	0.598	0.598	0.598	0.598	0.598	0.598	0.598

For heads over 100 ft. use $c = 0.598$. (From Hughes and Safford's "Hydraulics," First Edition, p. 146.)

The following data have been abstracted from Hughes and Safford's "Hydraulics," 1911, to which the reader is referred for a more complete discussion.

Procedure to Be Followed in Weir Measurements.—1. Constructing and setting up the weir and the gage for measuring the head; reproducing, if possible, the experimental conditions of the formula to be used.

2. Measuring the length of the crest and determining its irregularities if any.

3. Taking a profile of the crest if not sharp-edged.

4. Determining by actual measurements the cross-sectional area of the channel of approach.

5. Establishing by leveling the relative elevations of the crest of the weir and the zero of the gage.

6. When the desired regulation of flow is established, determining the head by hook gage or other observations at intervals as frequent as the conditions require.

7. If possible, measure actual velocity in the channel of approach by a current meter or some other direct method, and

8. Compute the discharge by the formula selected.

Three of these operations require especial consideration, *viz.*, construction and setting, the measurement of the head, and the selection of the formula.

Construction and Setting of Weirs.—1. A sharp-crested weir with complete crest contraction should be used.

2. The crest should be level, and its ends vertical.

3. The end contractions should be complete, or, if suppressed, entirely suppressed.

4. The upstream face should be vertical; the downstream so designed that the nappe has free overfall.

5. Free access for air under the nappe should be made certain.

6. The weir should be set at right angles to the direction of flow.

7. The channel of approach should be straight for at least 25 ft. above the weir, of practically uniform cross-section, and of slight slope (preferably none).

8. Screens of coarse wire or baffles of wood should be set in the channel, if necessary, to equalize the velocities in different parts of the channel, but not nearer the crest than 25 ft.

9. The channel of approach should have a large cross-sectional area in order to keep the velocity of approach low.

Measurement of Head.—The head above the crest of the weir should be measured, preferably, by a hook gage with vernier scale upon it, reading to thousandths of a foot.[1] Point gages and float gages may also be used, and, under certain circumstances, are advantageous.

For approximate results, the measurements to the water surface may be made from a peg driven into the bed of the stream at a distance

[1] In selecting a hook gage, choose one made entirely of metal, and with the slow-motion screw within easy reach of the observer when setting the point of the hook.

of several feet above and to one side of the weir. But for careful or precise measurements the gagings must be made in a still box, the locations of which should meet the following essential conditions.

1. The cross-sectional area of the communicating opening or pipe must be sufficient to allow free communication with the channel, even when throttled.

2. The channel end of this opening must be set into and exactly flush with the flat walls of the channel, or into a flat surface laid parallel to the direction of flow, and the pipe itself must be normal to the direction of flow.

3. The channel end of this opening must be located far enough upstream to avoid the slope of the surface curve, and not far enough to increase the observed head by the natural slope of the stream.

The area of increased pressure, which forms above the bottom, beginning at the upstream face of the weir and extending upstream, perhaps about to the beginning of the surface curve,[1] once thought to be a location at which the observed head would include the velocity head, has been proved to be a poor location for the opening.

Avoid perforated pipes, no matter where the holes are bored, laid transversely or longitudinally in the stream at different depths; avoid so-called piezometers of any form which project in any direction into the stream. After the Lowell hydraulic experiments were made, Francis sometimes used pipes with holes bored in a vertical plane in order to secure an average pressure across the stream, in recognition of the fact that the surface is not transversely level. Since his time, this has been shown to be a vicious practice, which may introduce more errors than it was designed to obviate.

The essential conditions of location of a still box will in general be met if its opening is set well upstream from the beginning of the surface curve, and at or a few inches below the crest level.

If Francis', Fteley and Stearns', Bazin's, or any particular experimenter's formula is to be used, his location should be duplicated (p. 200, Hughes and Safford's "Hydraulics").

Measurements of Head in Francis' Experiments.—The head was observed by two hook gages, one on each side of the channel, set in still boxes which were 18 in. long by 11 in. wide. Communication with the channel was made for the contracted weir measurements by a 1-in. diameter hole in the bottom of each box, located 6 ft. upstream from the weir and 4 in. lower than the level of the crest. For the suppressed weir, communication was established by pipes opening into the sides of the channel 1 ft. lower than the level of the crest, or by the single opening for the pipes which were set in the board . . . All three openings used were therefore 6 ft. upstream from the weir. To prevent rapid oscillations, the openings were throttled by a perforated plug set on the inside of the still boxes (p. 204, *loc. cit.*).

Measurements of Head in Fteley and Stearns' Experiments.— . . . The head was measured by hook gages set in still boxes which were connected with the channel by pipes. Although the actual form of piezometer openings varied, the essential condition that the opening be at or below the crest

[1] See Fteley and Stearns, *Trans. Am. Soc. C. E.*, 12, 42, Plate IV.

TABLE 38.—WEIR DISCHARGES DUE TO HEADS FROM 0.00 TO 2.99 FT.
Safford

Quantity of water in cubic feet per second, discharged over a weir with length of 1 ft., the weir to have complete contraction on its crest, and to have no end contractions. $Q = 3.31lH^{3/2} + 0.007l$ for depths up to 0.5 ft. and $Q = 3.33lH^{3/2}$ for depths above 0.5 ft. Head in feet; discharge in cubic feet per second.

H	*Q*	*H*	*Q*	*H*	*Q*	*H*	*Q*	*H*	*Q*
0.00	....	0.60	1.55	1.20	4.38	1.80	8.04	2.40	12.38
0.01	0.01	0.61	1.59	1.21	4.43	1.81	8.11	2.41	12.46
0.02	0.02	0.62	1.63	1.22	4.49	1.82	8.18	2.42	12.54
0.03	0.02	0.63	1.66	1.23	4.54	1.83	8.24	2.43	12.61
0.04	0.03	0.64	1.70	1.24	4.60	1.84	8.31	2.44	12.69
0.05	0.04	0.65	1.74	1.25	4.65	1.85	8.38	2.45	12.77
0.06	0.06	0.66	1.79	1.26	4.71	1.86	8.45	2.46	12.85
0.07	0.07	0.67	1.83	1.27	4.77	1.87	8.51	2.47	12.93
0.08	0.08	0.68	1.87	1.28	4.82	1.88	8.58	2.48	13.00
0.09	0.10	0.69	1.91	1.29	4.88	1.89	8.65	2.49	13.08
0.10	0.11	0.70	1.95	1.30	4.94	1.90	8.72	2.50	13.16
0.11	0.13	0.71	1.99	1.31	4.99	1.91	8.79	2.51	13.24
0.12	0.14	0.72	2.03	1.32	5.05	1.92	8.86	2.52	13.32
0.13	0.16	0.73	2.08	1.33	5.11	1.93	8.93	2.53	13.40
0.14	0.18	0.74	2.12	1.34	5.16	1.94	9.00	2.54	13.48
0.15	0.20	0.75	2.16	1.35	5.22	1.95	9.07	2.55	13.56
0.16	0.22	0.76	2.21	1.36	5.28	1.96	9.14	2.56	13.64
0.17	0.24	0.77	2.25	1.37	5.34	1.97	9.21	2.57	13.72
0.18	0.26	0.78	2.29	1.38	5.40	1.98	9.28	2.58	13.80
0.19	0.28	0.79	2.34	1.39	5.46	1.99	9.35	2.59	13.88
0.20	0.30	0.80	2.38	1.40	5.52	2.00	9.42	2.60	13.96
0.21	0.33	0.81	2.43	1.41	5.57	2.01	9.49	2.61	14.04
0.22	0.35	0.82	2.47	1.42	5.63	2.02	9.56	2.62	14.12
0.23	0.37	0.83	2.52	1.43	5.69	2.03	9.63	2.63	14.20
0.24	0.40	0.84	2.56	1.44	5.75	2.04	9.70	2.64	14.28
0.25	0.42	0.85	2.61	1.45	5.81	2.05	9.77	2.65	14.36
0.26	0.45	0.86	2.66	1.46	5.87	2.06	9.85	2.66	14.45
0.27	0.47	0.87	2.70	1.47	5.93	2.07	9.92	2.67	14.53
0.28	0.50	0.88	2.75	1.48	6.00	2.08	9.99	2.68	14.61
0.29	0.52	0.89	2.80	1.49	6.06	2.09	10.06	2.69	14.69
0.30	0.55	0.90	2.84	1.50	6.12	2.10	10.13	2.70	14.77
0.31	0.58	0.91	2.89	1.51	6.18	2.11	10.21	2.71	14.86
0.32	0.61	0.92	2.94	1.52	6.24	2.12	10.28	2.72	14.94
0.33	0.63	0.93	2.99	1.53	6.30	2.13	10.35	2.73	15.02
0.34	0.66	0.94	3.03	1.54	6.36	2.14	10.42	2.74	15.10
0.35	0.69	0.95	3.08	1.55	6.43	2.15	10.50	2.75	15.19
0.36	0.72	0.96	3.13	1.56	6.49	2.16	10.57	2.76	15.27
0.37	0.75	0.97	3.18	1.57	6.55	2.17	10.64	2.77	15.35
0.38	0.78	0.98	3.23	1.58	6.61	2.18	10.72	2.78	15.43
0.39	0.81	0.99	3.28	1.59	6.68	2.19	10.79	2.79	15.52
0.40	0.84	1.00	3.33	1.60	6.74	2.20	10.87	2.80	15.60
0.41	0.88	1.01	3.38	1.61	6.80	2.21	10.94	2.81	15.69
0.42	0.91	1.02	3.43	1.62	6.87	2.22	11.01	2.82	15.77
0.43	0.94	1.03	3.48	1.63	6.93	2.23	11.09	2.83	15.85
0.44	0.97	1.04	3.53	1.64	6.99	2.24	11.16	2.84	15.94
0.45	1.01	1.05	3.58	1.65	7.06	2.25	11.24	2.85	16.02
0.46	1.04	1.06	3.63	1.66	7.12	2.26	11.31	2.86	16.11
0.47	1.07	1.07	3.69	1.67	7.19	2.27	11.39	2.87	16.19
0.48	1.11	1.08	3.74	1.68	7.25	2.28	11.46	2.88	16.27
0.49	1.14	1.09	3.79	1.69	7.32	2.29	11.54	2.89	16.36
0.50	1.18	1.10	3.84	1.70	7.38	2.30	11.61	2.90	16.44
0.51	1.21	1.11	3.89	1.71	7.45	2.31	11.69	2.91	16.53
0.52	1.25	1.12	3.95	1.72	7.51	2.32	11.77	2.92	16.62
0.53	1.28	1.13	4.00	1.73	7.58	2.33	11.84	2.93	16.70
0.54	1.32	1.14	4.05	1.74	7.64	2.34	11.92	2.94	16.79
0.55	1.36	1.15	4.11	1.75	7.71	2.35	12.00	2.95	16.87
0.56	1.39	1.16	4.16	1.76	7.77	2.36	12.07	2.96	16.96
0.57	1.43	1.17	4.21	1.77	7.84	2.37	12.15	2.97	17.04
0.58	1.47	1.18	4.27	1.78	7.91	2.38	12.23	2.98	17.13
0.59	1.51	1.19	4.32	1.79	7.97	2.39	12.30	2.99	17.22

in and normal to a flat surface parallel to the direction of flow, was in all cases maintained. The location of each opening is stated in the table (p. 208, *loc. cit.*).

The General Weir Formula may be expressed by the equation $Q = clH^{3/2}$. To this form all the equations in use may be reduced, but it is better practice, in view of the several methods of correcting for the velocity of approach followed by the various experimenters, to use their forms of equation.

The Francis Weir Formula.

Let H = the observed head corrected to include the effect of the velocity of approach

h = the observed head upon the crest of weir, being the difference in elevation in feet between the top of the crest and the surface of the water in the channel, at a point upstream, which should, if possible, be taken just beyond the beginning of the surface curve

h_v = the head due to the mean velocity of approach

$= V_A{}^2/2g$

For contracted weirs, neglecting velocity of approach,

$$Q = 3.33(l - 0.1Nh)h^{3/2}$$

Note.—The use of h instead of H in the factor $(l - 0.1NH)$ used in correcting for end contractions is as precise as ordinary practice warrants.

For contracted weirs, head corrected for velocity of approach:

$$Q = 3.33(l - 0.1NH)[(h + h_v)^{3/2} - h_v{}^{3/2}]$$

For suppressed weirs, neglecting velocity of approach,

$$Q = 3.33lh^{3/2}$$

For suppressed weirs, head corrected for velocity of approach:

$$Q = 3.33l[(h + h_v)^{3/2} - h_v{}^{3/2}]$$

The Fteley and Stearns Formula.

$$Q = 3.31lH^{3/2} + 0.007l$$
$$H = (h + 1.50h_v) \text{ for suppressed weirs}$$
$$H = (h + 2.05h_v) \text{ for contracted weirs}$$

For contracted weir use $(l - 0.1NH)$ instead of l.

The King Formula,[1] based upon the experiments of Francis, Fteley and Stearns, and Bazin, is

$$Q = 3.34lH^{1.47}\left(1 + 0.56\frac{H^2}{d^2}\right)$$

where $d = \frac{A}{l}$, A being the area of the channel of approach.

For contracted weir use $(l - 0.1NH)$ instead of l.

Lyman's Diagrams and Tables[2] are based upon the experiments of Francis, Fteley and Stearns, and Bazin, and at the hydraulic laboratories of Cornell University and the University of Utah, and are in convenient form for use.

[1] KING, "Handbook of Hydraulics," 66.

[2] LYMAN, R. R., *Trans. Am. Soc. C. E.*, 1914; **72**, 1189.

The Francis formulas are strictly applicable only to vertical sharp-crested rectangular weirs with complete contractions and with free overfall and
when the head (H) is not greater than one-third the length (l);
when the head is not less than 0.5 ft. nor more than 2 ft.;
when the velocity of approach is 1 ft. per second or less;
when the height of the weir is at least three times the head.

In all probability the formulas are usable with higher heads than 2 ft., but not much lower than 0.5 ft., as shown by Fteley and Stearns' experiments.

Rehbock's Formula,[1] first published in Germany in 1911, and revised in 1912, is as follows:

$$Q = \frac{2}{3}\mu\sqrt{2g}\,lh^{3/2}$$

where

$$\mu = 0.605 + \frac{1}{1050h - 3} + 0.08\frac{h}{Z} \text{ (metric units)}^{3}$$

This formula is based upon numerous experiments made on sharp-crested, fully aerated weirs without end contractions, for values of

$$Z = 0.41,\ 0.66,\ 0.82, \text{ and } 1.64 \text{ ft.}$$
$$h = 0.03 \text{ to } 0.59 \text{ and } 0.75 \text{ ft.}$$
$$l = 1.64 \text{ ft.}$$

He extended the curves for μ beyond the range of his actual experiments by the law of hydraulic similitude to include values of

$$Z = 0.33 \text{ to } 6.56 \text{ ft.}$$
$$h = 0.03 \text{ to } 3.28 \text{ ft.}$$
$$\frac{h}{Z} = 0 \text{ to } 1.0 \text{ (preferably not over 0.8)}$$

Professor Schoder[2] states that Rehbock's formula gives values which agree more closely with a large mass of recent experimental data than the formulas of Fteley and Stearns, Bazin, and Francis, the error increasing with h and with Z. Francis' formula checked closely at low heads and low velocities of approach. Rehbock's formula, while comparatively unknown in this country, is largely used in Germany.

Choice of Formulas.—When Francis' weir settings can be duplicated or the velocity of approach is very low, 1 ft. per second or less, there is general willingness on the part of both engineers and laymen to accept this formula for heads for from 0.5 to 2 ft., and the same is true of the Fteley and Stearns formula for heads of 0.07 to 0.5 ft. For higher heads the Cornell experiments, which are the only guides, indicate that the Francis formula may be used with reasonable accuracy up to heads of 5 ft.

Bazin's formula is the best where his conditions can be reproduced and if the velocity of approach is high and the height of weir low, his formula is the only one sufficiently flexible. For this reason it is the most useful.

Smith's coefficients are the result of the most thorough study, but are based upon experimental data of somewhat unequal accuracy. They do, however, furnish means for satisfactory interpolation to suit cases not covered precisely by the data which he used.

[1] REHBOCK, TH., *Proc. Am. Soc. C. E.* Aug., 1928; 1771.

[2] SCHODER, ERNEST W., and KENNETH B. TURNER, "Precise Weir Measurements," *Proc. Am. Soc. C. E.*, Sept., 1927; 1395.

[3] In English units $\mu = 0.605 + \frac{1}{320h - 3} + 0.08\frac{h}{Z}$.

If possible, contracted weirs should be avoided, but are often necessary to insure atmospheric pressure underneath the nappe; if end contractions are unavoidable, the Francis formula should be used.

For rough measurements there has never appeared to be any good reason for departing from the Francis formula, which has the advantage of long usage and consequent familiarity, especially in legal cases, although it has often been used far beyond the limits laid by Francis himself. It should be borne in mind, however, that his formula applies only to sharp-crested weirs (p. 223, *loc. cit.*).

Exception should be made, however, of cases where the head is less than 0.5 ft., for which the Francis formula is not applicable. Table 38 on page 153 may be taken as a reasonable guide, having been computed by the Francis formula for heads greater than 0.5 ft. and by the Fteley and Stearns formula for lesser heads. No formula is safely applicable to extremely low heads. For suppressed weirs, precautions should always be taken to secure adequate ventilation so as to insure atmospheric pressure underneath the nappe, or formulas will not be applicable.

Schoder and Turner[1] have made a comprehensive study of the accuracy of various formulas for determining the discharge over sharp-crested weirs and find an important source of error in such formulas to be in their failure to properly allow for velocity of approach and in unequal horizontal distribution of velocities. They suggest a formula which includes velocity head of approach for the section above and below the elevation of the weir crest, considered separately.

Submerged Weirs.—When the water surface in the channel below the weir is higher than the crest, the weir is said to be submerged or drowned. Measurements by submerged weirs are much less certain than by weirs with free discharge, but their use is sometimes unavoidable.

Experiments on submerged weirs have been made by Francis, Fteley and Stearns, and Bazin, but the number is comparatively small. Formulas have been proposed by Francis, Fteley and Stearns, Herschel, Bazin and King. None of these can be considered as altogether satisfactory.

King has suggested[2] the formula

$$Q = l\sqrt{h}(3.4h + 4.4h_2)$$

where h_2 is the depth of submergence, or height of the water above the crest on the downstream side, and $h = h_1 - h_2$ or net head on weir. The coefficients in this formula are based upon the experiments of Francis and Fteley and Stearns, but there is no logical method of correcting for velocity of approach or applying other corrections.

The formulas proposed by Fteley and Stearns, and Herschel, require the use of variable coefficients, and tables of such coefficients are necessary for their application.

[1] Schoder, Ernst W., and Kenneth B. Turner, "Precise Weir Measurements," *Trans. Am. Soc C. E.*, 1929; **93**, 999.

[2] King and Wisler, "Hydraulics," p. 119.

The Bazin Formula is

$$Q = \left(1.05 + 0.21\frac{h_2}{Z}\right)\left(\frac{h^{1/3}}{h_1}\right)\left(0.405 + \frac{0.00984}{h_1}\right)\left(1 + 0.55\frac{h_1^2}{d'^2}\right)lh_1\sqrt{2gh_1}$$

where

h_1 = head on weir of upstream water surface
h_2 = head on weir of water surface down stream
$h = h_1 - h_2$
d' = depth of water above weir $= \frac{A_1}{l}$
d'' = depth of water below weir $= \frac{A_2}{l}$
A_1, A_2 = area of cross-section of channel above and below weir
l = length of weir crest
Z = height of weir crest above bottom of channel

King[1] has also devised the empirical formula.

$$Q = 3.34lh^{1.47}\left(1 + 0.56\frac{h_1^2}{d'^2}\right)$$

All the foregoing discussion and formulas apply to weirs without end contractions. When there are end contractions, approximate results may be obtained by using $l - 0.1NH$ for l. Comparisons by King show that his formula agrees well with the results of all available experiments.

The King formula is somewhat difficult of computation, but it may be applied with comparative ease by means of the tables given in King's "Handbook of Hydraulics," 2d ed., pp. 131, 135, and 136.

Triangular Weirs.—The theoretic discharge of the triangular weir is given by the equation, $Q = \frac{4}{15}B(2g)^{1/2}H^{3/2}$
in which

B = length of base of triangle at level of H or water surface
H = head over angle of the weir notch in feet

Experiments on flow over triangular weirs in which the angle is a right angle, which is the only form used in practice, have been made by Thomson; by Barr; at the University of Michigan; and at the Massachusetts Institute of Technology. Summarized results of the first three sets of experiments are given by King in his "Handbook of Hydraulics," 2d ed., p. 93. Thomson's experiments covered a range of heads from about 0.15 to 0.60 ft.; Barr's from about 0.15 to 0.85; and the University of Michigan's, from about 0.15 to 1.8. Their results may fairly be expressed by the following equations.

Thomson's experiments, $Q = 2.54H^{5/2}$
Barr's experiments, $Q = 2.48H^{2.48}$
University of Michigan's experiments, $Q = 2.52H^{2.47}$

[1] "Handbook of Hydraulics," 2nd Ed., p. 87.

Experiments made at the Massachusetts Institute of Technology, under the direction of Professor Dwight Porter, gave for the right-angled notch weir,

$$Q = 2.54H^{5/2}$$

Table 39 shows the values of Q computed by the Thomson and King formulas. The differences are not of much consequence and either formula may probably be applied with a fair degree of satisfaction. Since the King formula is the only one which is based on experiments with heads greater than 1.0 ft., it is probably more trustworthy under such conditions.

TABLE 39.—DISCHARGE OF RIGHT-ANGLED TRIANGULAR WEIR, BY THOMSON FORMULA $Q = 2.54H^{5/2}$ AND BY KING FORMULA $Q = 2.52H^{2.47}$ [1]

Head, feet	Discharge in cubic feet per second	
	Thomson formula	King formula
0.1	0.0080	0.0085
0.2	0.0452	0.0473
0.3	0.125	0.129
0.4	0.257	0.262
0.5	0.449	0.455
0.6	0.708	0.714
0.7	1.041	1.044
0.8	1.454	1.452
0.9	1.951	1.943
1.0	2.540	2.520
1.1	3.223	3.189
1.2	4.007	3.954
1.3	4.893	4.818
1.4	5.890	5.785
1.5	7.000	6.860

[1] For detailed table of discharge by King's formula, see "Handbook of Hydraulics," 2d ed., p. 141.

Trapezoidal Weirs.—The trapezoidal weir differs from the rectangular type in that the sides are inclined rather than vertical. Usually the sides are given a batter of 1 horizontal in 4 vertical for the reason that at this angle the slope is just about sufficient to offset the effect of end contractions. When this is done the weir is known as the "Cippoletti weir." The general equation for the trapezoidal weir is as follows:

$$Q = \tfrac{2}{3}(2g)^{1/2}lH^{3/2} + \tfrac{4}{15}2z(2g)^{1/2}H^{5/2}$$

in which z = the batter of the side or the ratio of the vertical projection to the horizontal projection of the side.

For the Cippoletti weir in which $z = \frac{1}{4}$, the formula reduces to

$$Q = 3.367lH^{3/2}$$

Other Weirs.—For the determination of the discharge over broad-crested weirs and dams having different types of crests, reference may be had to "Weir Experiments, Coefficients, and Formulas" by Robert E. Horton;[1] King's "Handbook of Hydraulics"; Williams and Hazen's "Hydraulic Tables"; and various books on hydraulics.

Side weirs or spillways parallel to the direction of flow in a channel are not measuring devices. They are utilized in regulating devices, and are discussed in Chaps. II and XVI.

Venturi-type Meter.—The principle of this apparatus, based upon Bernoulli's theorem, was discovered about 1791 by the Italian engineer,

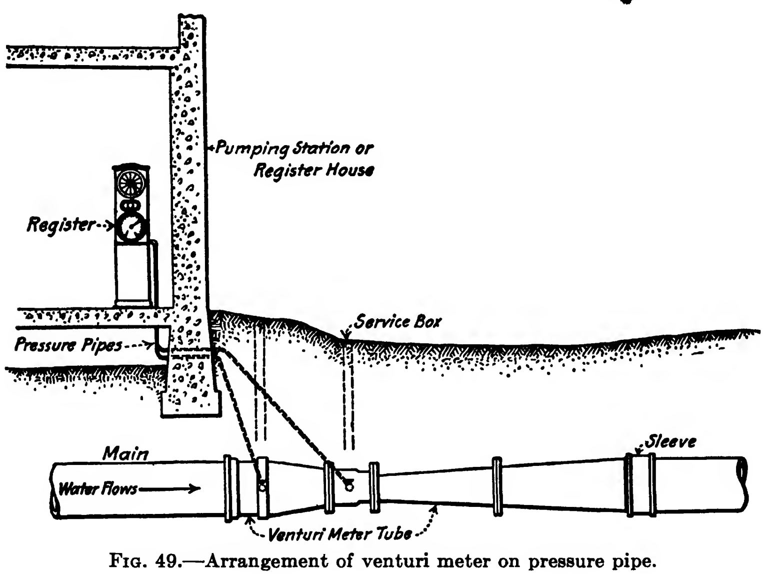


FIG. 49.—Arrangement of venturi meter on pressure pipe.

J. B. Venturi, who stated that, when fluids or gases are discharged through an expanding nozzle, a sucking action is exercised in the small diameter, diminishing as the diameter increases. This principle was first practically applied by Clemens Herschel in 1887 in the so-called Venturi meter. The meter tube, which is the portion of the apparatus to which Venturi's discovery applies, is inserted in a line of pipe under pressure and consists of three parts, the inlet cone, in which the diameter of the pipe is gradually reduced, the throat or constricted section, and the outlet cone, in which the diameter increases gradually to that of the

[1] Published as Water Supply and Irrigation Paper 200 of the U. S. Geological Survey, 1907.

pipe in which the meter is inserted. The throat is lined with bronze; its diameter, in standard meter tubes, is from one-third to one-half of the diameter of the pipe; and its length but a few inches, sufficient to allow a suitable pressure chamber or piezometer ring to be inserted in the pipe at this point. The upper or inlet cone has a length of approximately one-fourth that of the lower cone. A piezometer ring is inserted at the upper or large end of the inlet cone and the determination of the quantity of water flowing is based upon the difference in pressures observed or indicated at this point and at the throat of the meter. The general form of the meter is shown in Fig. 49.

The derivation of a formula from which the discharge of the Venturi meter tube is computed may be found in Hughes and Safford's "Hydraulics," First Edition, p. 116. As written by Herschel, the form of this expression is

$$Q = \frac{A_1A_2}{\sqrt{A_1^2 - A_2^2}}\sqrt{2g(h_1 - h_2)}$$

$$= \frac{A_1A_2}{\sqrt{A_1^2 - A_2^2}}\sqrt{2gH}$$

in which A_1 and A_2 are the areas in square feet at the upstream end and at the throat of the meter, respectively, h_1 and h_2 the pressure heads at the corresponding points, and

$$H = h_1 - h_2$$

Under actual operating conditions, and for standard meter tubes, including allowance for friction, this formula reduces to the form

$$Q = (1.00 \pm 0.02)A_2\sqrt{2gH}$$

The coefficient written (1.00 ± 0.02) is made up of two parts, or $c = c_1c_2$.

$$c_1 = \frac{A_1}{\sqrt{A_1^2 - A_2^2}}$$

$$c_2 = \text{coefficient of friction}$$

For standard meter tubes in which the diameter of the throat is between one-third and one-half that of the pipe, the values of c_1 range between 1.0062 and 1.0328, while the friction coefficient c_2 varies from 0.97 to 0.99. Thus the range of values of c is from 0.98 to 1.02, and accordingly c has been written above as (1.00 ± 0.02). Hazen[1] thinks $c = 0.99$ the best value for practical use.

J. W. Ledoux[2] gives results of experiments made by him on 19 meter tubes which show a coefficient approximating 0.977 for ordinary velocities and falling as low as 0.90 for a velocity of 1.0 ft. per second through

[1] *Eng. News*, 1913; **70**, 199.

[2] "Venturi Tube Characteristics," *Trans. Am. Soc. C. E.*, 1927; **91**, 565.

the throat of the tube. Ledoux[1] uses the formula for velocity at the upstream end

$$v_1 = c\sqrt{\frac{2gH}{\left(\frac{d_1}{d_2}\right)^4 - 1}}$$

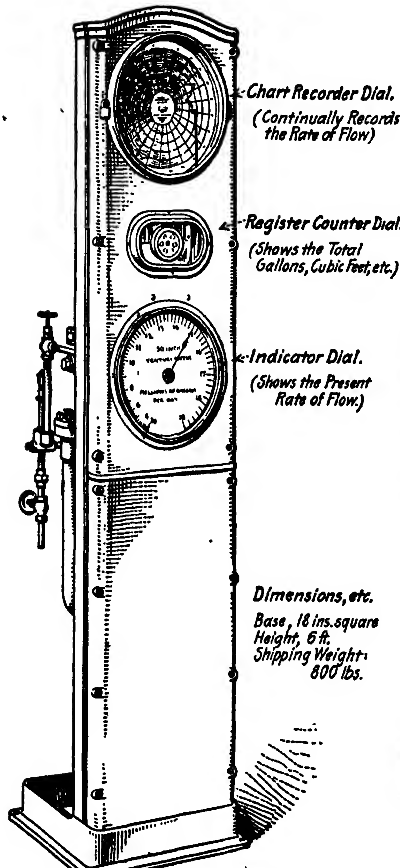


Fig. 50.—Type M register-indicator recorder.

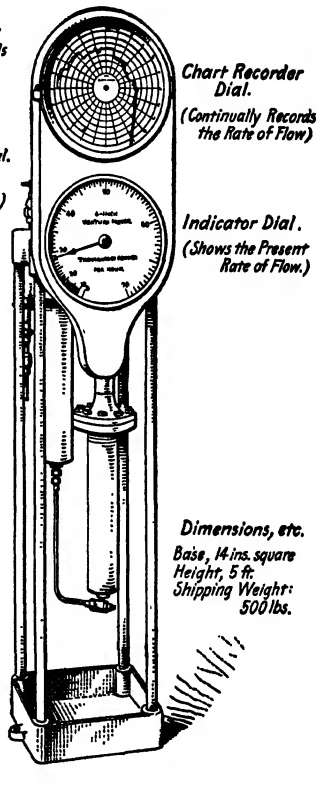


Fig. 51.—Type M indicator recorder.

and finds for the loss of head h_f in the meter tube

$$h_f = \frac{v_2^{1.961}}{403}$$

The relations between heads (both the Venturi head and the total loss of head in the tube) and the quantities flowing or velocities are shown graphically in a simple diagram by R. T. Regester.[1] This is for standard tubes of the Builders Iron Foundry pattern, and is stated to be correct within about 3 per cent.

The Venturi meter affords one of the most accurate methods of measuring water in closed pipes under pressure, the registration being within 2 per cent of the actual flow of water at ordinary velocities.

The Venturi meters are made by Builders Iron Foundry and by the Simplex Valve & Meter Co. The indicating and recording instruments made by Builders Iron Foundry are illustrated in Figs. 50 and 51. The principle of the Simplex recording device was described by J. W. Ledoux.[2]

The *flow meter* of the Bailey Meter Company makes use of the Venturi principle, but ordinarily employs a nozzle inserted in the pipe

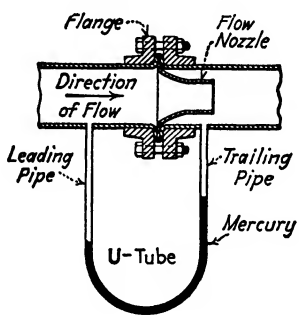


FIG. 52.—Illustration of the principle of operation of a flow meter employing a nozzle to cause a difference in pressure.

instead of the Venturi tube. Figure 52 illustrates the form of the nozzle and the method of inserting it in a pipe and shows how the difference in pressure may be obtained. The throat of the nozzle is considerably larger in diameter than the throat of the Venturi tube for the same rate of flow, and the resulting total loss of head is approximately the same for either.

The Bailey Meter Company has two types of indicating and recording instruments for use with the nozzle. One is electrically operated, in which the differential gage measuring the difference in pressure above and at the nozzle regulates the amount of alternating current flowing in an electrical circuit, and this actuates the indicating and recording instruments, which may be at any distance from the nozzle. The other is mechanically operated by a float on the mercury

[1] Published in *Eng. News-Rec.*, 1926; **97**, 717.

[2] *Trans. Am. Soc. C. E.*, 1913; **76**, 1048.

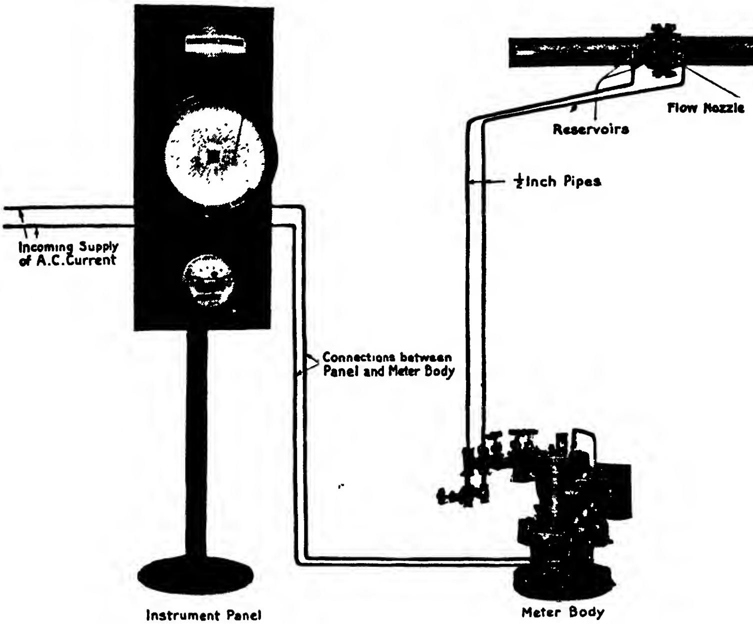


FIG. 53.—Indicating, recording, registering flow meter, electrically operated. (*Bailey Meter Co.*)

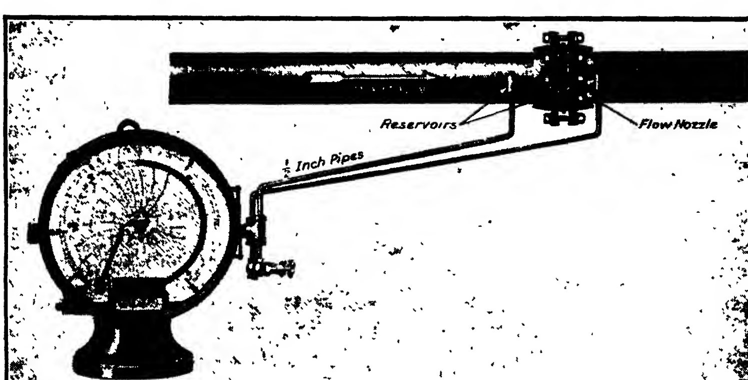


FIG. 54.—Indicating, recording, registering flow meter, mechanically operated. (*Bailey Meter Co.*)

or other liquid of the differential gage and must be fairly near the nozzle in order that the pressure pipes may function satisfactorily, as in the case of the meters employing the Venturi tube. Figures 53 and 54 show the general appearance of the electrically and mechanically operated flow meters, respectively.

For the measurement of sewage, the Bailey Meter Company has adapted only the electrically operated meter. In this case the regulation of the quantity of current flowing is accomplished by two copper floats in vertical iron pipes receiving the pressure through piezometer piping from either side of the flow nozzle. The floats are suspended from a lever arm, which has a suitable connection through a crank and connecting rod with the rotor of an induction element by means of which the current is regulated. The receiving and recording instrument is the same as that shown in Fig. 54.

The piezometer connections to the float tubes are made of 1½-in. pipe, with connections for clean water under pressure, by means of which any accumulation of solids can be forced back into the sewage main. There is also a connection by which a trickling stream of clean water may be allowed to flow into the float tubes, thus preventing the accumulation of sufficient solids to interfere with the operation of the floats. Hand holes for cleaning should be provided in the main, on either side of the nozzle.

Other makers of instruments, among whom are the Republic Flow Meters Co. and the Brown Instrument Co., manufacture flow meters, many, if not all, of which are applicable for use with Venturi tubes, nozzles, orifices, or Pitot tubes inserted in the conduits. Pitot tubes would be unsuited to measuring sewage, and it is probable that thin plate orifices would prove unsatisfactory. It is probable that with proper precautions for sealing and cleaning the pressure tubes and piezometric openings, any of these instruments could be used for recording sewage flow, but they have not yet been so utilized to any considerable extent.

In the Venturi meter used for measuring sewage, at each annular chamber or piezometer ring there should be valves by which the pressure openings can be closed, and these valves may be so designed that in closing a rod is forced through the opening so as to clean out effectually any matter which may have clogged it. When all four of these valves have been closed the plates covering the hand holes in the pressure chamber may be removed and the chamber cleaned by flushing with hose or otherwise. Such flushing at short intervals is usually necessary if Venturi meters for sewage are to be maintained in good operating condition.

In order to prevent the interference with the operation of the register by clogging, an oil seal may be inserted in the pressure pipe, between the

meter tube and the register. The pressure is transmitted as far as the seal, through water in the pressure pipes, and from the seal to the register through oil. Thus it is impossible for any sewage to get into the register and interfere with its proper operation. Such an oil seal is shown in Fig. 55, which illustrates the apparatus at the Ward Street Pumping Station of the Boston Metropolitan Sewerage Works. The register shown in this figure is of an old type, not like those illustrated in Figs. 50 and 51.

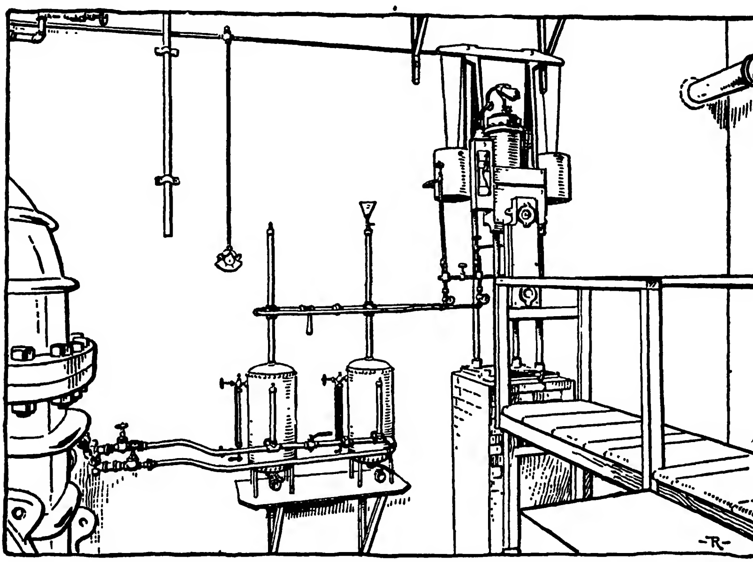


FIG. 55.—Oil seal and Venturi register, Ward Street Pumping Station, Boston.

Venturi Flume.—Since the Venturi type meter is only applicable to closed pipes under pressure, it can be used for measuring sewage only in force mains or inverted siphons.

In recent years, the Venturi principle has been utilized for the measurement of flow of water in open channels by means of the Venturi flume, in which the side walls and in some cases the bottom of the channel are so shaped as to approximate the form of the Venturi tube.

This apparatus has been used principally in connection with the distribution of water in irrigation ditches, but its use as a device for measuring sewage is being introduced in several cities. A flume of this type is used to measure the flow of sewage reaching the Syracuse Sewage Works, and another was placed in operation on East Forty-first Street by the Borough of Manhattan in November, 1926. The latter is illustrated by Fig. 56 and is of yellow pine, anchored to the side walls

of an 8- by 8-ft. box sewer, the arch of which was raised 4 ft. above the flume to allow for gagings during extremely high flows. The capacity, limited by the recording device to a depth of 10 ft. at the upstream piezometer pipe, is 343 cu. ft. per second. When operating at maximum capacity, the incoming sewer flows with a surcharge of about 2 ft.

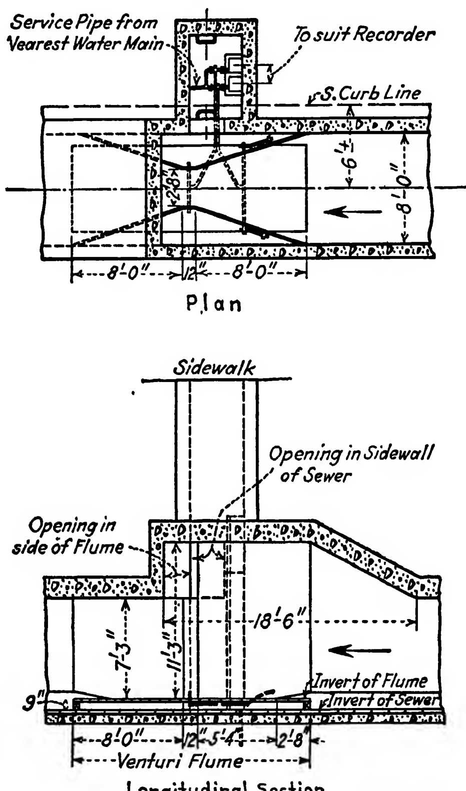


FIG. 56.—Venturi flume inserted in East 41st street sewer, Manhattan, New York.

Three-inch brass piezometer pipes connect throat and upstream sections with concrete stilling wells built in an offset manhole. A float-operated Au recorder is installed in an operating chamber at the top of the offset manhole. Water connections 1½ in. diameter with control valves in the operating chamber are provided for blowing out the piezometer pipes. Kenneth Allen, Sanitary Engineer, Board of Estimate and Apportionment of the City of New York, stated that this was done once a week and up to September, 1927, no clogging had

occurred and the stilling wells had been kept quite free from deposit, while there had been an absence of surging that would show on the record.

There is, as yet, comparatively little experimental information available on Venturi flumes. P. S. Wilson and C. A. Wright[1] give the results of a series of experiments at Cornell University. Their measurements were made in a 24-in. flume, in which a throat 8 in. wide was formed, the contraction taking place in a length equal to three times the width of the throat, while the divergence was accomplished in a distance equal to ten times the throat width. They used two throats, one having a length of 8 in., the other 24 in. The throat gages were tapped into the sides at a distance from the upstream end equal to two-thirds the length of the throat. The discharge coefficients obtained for the throat 8 in. long varied in an erratic manner, but those for the throat 24 in. long were reasonably consistent, and varied between 0.96 and 1.00, averaging about 0.98.

The conclusions of Wilson and Wright were:

1. The coefficient of the Venturi flume for any particular condition of flow within rather wide limits is a fixed determinable quantity about unity. With the flumes experimented upon, however, the value of the coefficient varied over a range of possibly 10 per cent with varied conditions of flow, most of this variation being apparently due to surface phenomena, waves, etc., the effect of which could be controlled by the design of the flume so as to obtain a more definite coefficient.

2. The maximum loss of head necessary in the use of the Venturi flume is very small compared to that lost with other measuring devices under similar conditions. A weir with the same range of capacity would probably involve at least five or six times as much loss of head.

3. The necessity for connecting each of the gage wells to a pair of opposite gage openings in the channel was verified.

4. The exact location of the upstream gage openings was found to be immaterial as long as they were placed upstream from, and fairly close to the beginning of the convergence.[2]

The earliest experiments with the Venturi flume were those of V. M. Cone, made at Ft. Collins, Colo., for the U. S. Department of Agriculture.[3] All of the throat sections were 1 ft. in length, with widths varying from 1 to 7 ft. The contracting and expanding sections were identical, each having a length equal to three times the width of the throat. The throat gage was located at the middle of one side, and the upstream gage was in the side wall of the contracting section at a distance above

[1] *Eng. News-Rec.* 1920; **88**, 452.

[2] It is to be noted that other experimenters have located the upstream gage in the converging section, instead of above it—presumably because such location was more accessible and the insertion of a Venturi flume in an existing conduit is greatly simplified if it is not necessary to provide gage openings above the flume.

[3] *Jour. Agric. Res.*, Apr. 23, 1917.

the throat equal to twice the throat width. Cone gave diagrams of discharge for throats 1, 1½, and 2 ft. wide. These flumes were not as well shaped as those tested by Wilson and Wright, and the gages were not as well placed and were on one side only.

Further experiments have been made on flumes of the same proportions at Cornell University and on the Cache la Poudre River near Ft. Collins, Colo. The results are discussed in *Bull.* 265 of the Agricultural Experiment Station of the Colorado Agricultural College at Ft. Collins, by Ralph L. Parshall and Carl Rowher. In these later experiments the heads were observed on both sides of the converging section and of the throat and it was found that errors as large as 10 or 12 per cent might be introduced in the case of a throat width of 5 ft., by reading the heads on one side only. With smaller flumes, the errors from this cause decreased, and became comparatively insignificant for a throat 1 ft. wide.

From the calibration tests of flumes with throats 1, 1½, 2, 3, and 5 ft. in width, and with discharges ranging from less than 1 to nearly 400 cu. ft. per second, Parshall and Rowher derived the formula for this type of flume

$$Q = cbh_2 \sqrt{\frac{2gh}{1 + \frac{9}{49}\left(\frac{h_2}{h_1}\right)^2}}$$

in which Q = discharge in cubic feet per second
b = width of throat in feet
h_2 = head at throat in feet
h_1 = head in converging section in feet
$h = h_1 - h_2$
c = a coefficient, the value of which can be obtained from the equation

$$c = 0.9975 - 0.0175b + \frac{(h - 0.163h_1^{1/2})^2(20 - b)}{8h_1^2}$$

A comparison of 453 actual measurements with the results computed by this formula showed that 55 per cent agreed within 2 per cent; 68 per cent within 3 per cent; and 88 per cent within 5 per cent. The formula may, therefore, be considered sufficiently accurate for most practical purposes.

Improved Venturi Flume.—*Press Bull.* 60 of the Agricultural Experiment Station of the Colorado Agricultural College, dated January, 1925, by Ralph L. Parshall, announces the design of an improved Venturi flume, of the proportions shown in Fig. 57, in which the discharge is a function of the width of the throat and the head in the converging section only, and that these conditions obtain up to depths of sub-

mergence (at the end of the converging section) of nearly 75 per cent of the depth at entrance. The formula for the discharge of this flume is

$$Q = 4bh_1^{1.522b^{0.026}}$$

Since the throat width is a constant, the discharge can be obtained from a single measurement and it is possible to graduate a tape attached to a float in the gage well so as to show directly the quantity flowing. One hundred fifty-two experiments have been made on this flume at Ft. Collins, with throats 1 to 8 ft. wide, and with discharges from 0.05 to 60 cu. ft. per second; 88 per cent of them agreed with the formula within 3 per cent.

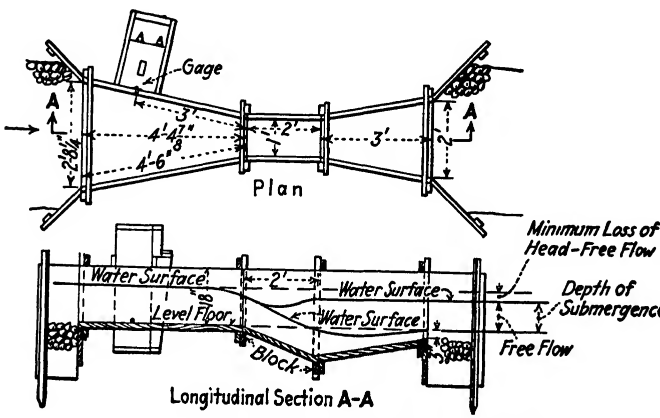


FIG. 57.—Improved Venturi flume. (*Parshall.*)

Further data are given in a paper by Parshall.[1] The total loss of head in a 3-ft. flume of this pattern is reported as 0.19 ft. for a discharge of 9.48 c.f.s. In discussing this paper, H. B. Muckleston calls attention to the fact that such a device, where but one head is measured, is really a special form of weir, rather than a true Venturi flume.[2] He also gives a simple form of diagram for the solution of the formula for discharge; and C. E. Carter shows that for the data presented, the formula

$$Q = 3.90bh_1^{1.58}$$

gives results quite as consistent with those obtained by tests as does the more complex formula presented by Parshall.

Tests of Improved Venturi Flumes with throat widths from 1 to 10 ft. are described in a discussion by Carl Rohwer in *Proc. Am. Soc. C. E.*, April, 1928, 1269.

[1] *Trans. Am. Soc. C. E.*, 1926; **89**, 841.

[2] Julian Hinds suggests the name "control-section meter."

Current meter measurements may be employed for the accurate determination of the velocity of flow in sewers of considerable size or in open channels, provided there be not too much paper or other suspended matter likely to clog the meter. The current meter must be calibrated by moving it at a uniform speed in still water. Knowing the constant or rating of the meter, the average velocity of the water at the point where it is held may be obtained with a high degree of accuracy.

Gagings of flow may be made by several methods, the one-point method, the two-point method, the multiple-point method, the method of integrating in sections, and the method of integrating in one operation.

In the one-point method, the meter is held at 0.6 of the depth and in the center of the stream, and the result is assumed to indicate the mean velocity of the stream. This is but a rough approximation, suitable only for hasty observations with no pretense to accuracy.

In the two-point method, the velocity is observed at 0.2 and 0.8 of the depth, and the average of these two figures is taken to represent the average velocity in the vertical section. The stream can be divided into a number of vertical sections, and the average velocity in each determined approximately by this method.

By the multiple-point method, the velocity at each of a large number of points, each representing the center of an equal area of the cross-section of the stream, is determined, and the average of the observed velocities is taken as the mean velocity in the section. Or, the velocities are observed at a large number of points, and lines of equal velocity in the cross-section are then drawn and measured by planimeter; by utilizing the method employed in computing mean elevation of a given area from a contour map the average velocity may be found. The employment of this method assumes a condition of steady flow, not only for the whole body of water but also for each filament, since it is obviously impossible to observe simultaneously the velocities at all points in the cross-section.

By the method of integrating in sections, the cross-section of the stream is divided into a number of vertical sections and the mean velocity in each is determined by lowering and raising the meter from top to bottom and back to the top of each section, at a uniform speed, for each observation. This is usually the most accurate and satisfactory method of making ordinary current meter gagings.

In integrating in one operation, the meter is lowered and raised as in integrating by sections, but at the same time is moved in a horizontal direction across the stream at a uniform rate. The result is intended to show the average velocity of the stream at one operation. With a skillful operator, results of a high degree of accuracy may be obtained by this method, and much more rapidly than by integrating in sections.

In a masonry conduit of regular form it is possible to make integrations in one operation by means of a track board and pivoted sleeve, by which the meter is guided so as to pass over the entire area of the section of the stream, and if it is moved at a uniform speed, results of great accuracy may be obtained in this way. This method is employed in gaging the flow in the aqueducts of the (Boston) Metropolitan Water Works, and has been described in detail by Walter W. Patch.[1]

An excellent discussion upon measurement of flow by meter observations will be found in Hughes and Safford's "Hydraulics," and in Hoyt and Grover's "River Discharge," 1908. The subject is also treated by J. C. Hoyt and N. C. Grover in certain of the "Water Supply Papers" of the U. S. Geological Survey.

Float measurements of the flow in sewers are rarely made except in rectangular channels or for the approximate determination of the velocity of flow between two manholes; but in studies of tidal currents or of sewage currents in bodies of water into which sewage may be discharged, floats are universally employed. Occasionally, however, the use of floats to measure the velocity of flow in comparatively small sewers is practicable, as, for instance, at Phoenix, Ariz., where a special float like a loosely fitting plunger was used in 24-, 30-, and 36-in. pipe sewers, and its time of passage noted.

Three types of floats may be used—surface floats, subsurface floats, and rod or spar floats. Only surface velocities can be obtained by the use of surface floats and the results can be considered only as approximations, owing to the modifying effects of the wind. Subsurface floats consist of relatively large bodies slightly heavier than water, connected by fine wires to surface floats of sufficient size to furnish the necessary flotation and carrying markers by which their courses may be traced. The resistance of the upper float and connecting wire is generally so slight that the combination may be assumed to move with the velocity of the water at the position of the submerged float. Rod floats have been used for measuring flow in open flumes, with a high degree of accuracy. They generally consist of metal cylinders so loaded as to float vertically. The velocity of the rod has been found to correspond very closely with the mean velocity of the water in the course followed by the float. Detailed descriptions of the methods of making accurate measurements of flow in rectangular flumes may be found in Francis' "Lowell Hydraulic Experiments" and in Hughes and Safford's "Hydraulics."

The use of dyes for measuring the velocity of flow in sewers, particularly in small-pipe sewers, is one of the simplest and most successful methods that has been used. Having selected a section of sewer in

[1] **"Measurement of the Flow of Water in the Sudbury and Cochituate Aqueducts,"**

which the flow is practically steady and uniform, the dye is thrown in at the upper end, and the time of its arrival at the lower end is determined. If a bright-colored dye is used, such as eosin, and a bright plate is suspended horizontally in the sewer at the lower end, the time of appearance and disappearance of the dye at the lower end can be noted with considerable precision, and the mean between these two observed times may be taken as representative of the average time of flow.

Chemical and electrical methods of gaging, while of great value in measuring clean water, are not so applicable to sewage measurements on account of the relatively large amounts of foreign matter contained in the sewage.

CHAPTER V

QUANTITY OF SEWAGE

Much information relating to the quantity of sewage likely to be, and actually being, produced by municipalities has been published in various papers and reports. As the quantity is of fundamental importance in the design of sewers, intercepters, pumping stations, and treatment works, an effort has been made herein to bring together some of the more significant data and to set forth the principal conditions influencing the volume of sewage.

Sources of Sewage.—The quantity of sewage which must be provided for may be considered as made up of definite portions of (1) domestic and industrial sewage, consisting of water from the public water supply carrying the waste products due to modern domestic and industrial conditions; (2) industrial waste waters not originating from the public water supply, consisting of certain quantities of water procured from other sources such as wells, rivers and lakes, defiled by the processes in which they have been used; (3) the water which finds its way into the sewers through infiltration and which is either ground water as ordinarily considered, or (in close proximity to rivers) may be water filtering through the ground from rivers; and (4) rainfall immediately collected and called "storm water." This last item is treated in Chaps. VII, VIII, IX, and X.

Relations between Quantity of Sewage, Population and Area Served.—The volume of domestic sewage per capita of population varies rather widely in different municipalities and in the several portions of each municipality. Such differences are due to varying proportions of the population actually tributary to the sewers and to diversity in customs and facilities for the use of water. Usually, the volume of domestic sewage is somewhat less than the corresponding volume of water consumed, because the water used for certain industrial purposes, for extinguishing fires and for sprinkling lawns, as well as that lost by leakage from the main pipes, will not appear in the domestic sewage.

The quantity of industrial sewage resulting from the use of the public water supply depends upon various conditions, and usually bears little if any relation to population. Nevertheless, the total quantity of sewage, both domestic and industrial, resulting from the use of the public water supply, usually bears a close relation to the population.

The volume of industrial waste waters not originating from the public supply depends upon the quantities of water available from other sources, the character of the industries and other local conditions. Data relating to such wastes can only be obtained from the plants producing them.

The volume of ground water entering sewers varies widely, and depends upon the extent to which the sewers are below ground-water level, the kind of sewers, the excellence of their construction, the character of soil in which they are laid, and various other conditions. In general, it will vary with the lengths of the sewers and with the areas tributary to them.

ECONOMIC PERIOD OF DESIGN

Sewerage works are generally so designed that they will be capable of serving the community adequately when it is of a size which it shall have reached after the lapse of a term of years called "the economic period of design." The determination of the economic period of design is based upon a consideration of the probable growth of the community, the difficulty of relieving the system when it shall have become overtaxed, and the inconvenience to the public caused by the construction of sewers in the streets, on the one hand; and on the other hand, the carrying charges for sewers having surplus capacity and the difficulties of maintenance due to small flows in large sewers.

Laterals and sub-main sewers are usually designed of adequate capacity to serve an indefinite period of time, that is, until the district shall have reached its assumed ultimate development; although there have been instances where radical changes in the character of a district have required extensive changes of such sewers. In the case of mains or trunk sewers of large size, particularly those intended to carry storm water, it may prove economical to base designs upon a comparatively short period of time, with the intention of relieving these sewers by constructing substantially parallel lines in other streets at a later date. In the design of intercepters, pumping stations, and treatment works, it is generally advisable to provide for a limited period of time which may be considerably shorter, in the case of such works as pumping stations and treatment plants, than in the case of intercepters. The latter cannot so readily be enlarged or relieved by means of parallel lines, but additions can be made to pumping stations and treatment plants, usually without material sacrifice of the portions previously built.

There are so many conditions which may affect the growth of a community that estimates of its future size and requirements are attended by much uncertainty. It is therefore usually inadvisable to predicate

designs upon estimates of conditions which may exist more than from 30 to 50 years in the future. It is possible to compute the "economic period" corresponding to the estimates of growth and requirements, but as such forecasts cannot be exact, precise computations of the economic period are not warranted and the engineer may as well determine the economic period directly by the exercise of judgment. Where structures are paid for by borrowed money it is generally considered wise that they should be of adequate size fully to serve their purpose at least until the maturity of the securities issued, although this policy need not prevent building of a pumping station, a treatment plant, or similar works, with a view to their extension within the period during which the securities for the portion first built are to run.

POPULATION

Growth.—It is impossible to forecast precisely the population of a city at any definite time in the future, or the rate at which the city will grow. A consideration of the growth of a city in the past, its location and natural advantages, together with a study of the past growth of other cities now of greater size, however, makes it possible to prepare a logical estimate of the probable future rate of growth.

The present population, if no recent census has been taken, may be estimated in a number of ways. The most obvious method is to assume that the rate of growth has been uniform and the same as that between the two most recent census enumerations. Where the number of "assessed polls" is known, it is possible to obtain a fair approximation of the total population by multiplying this figure by a factor obtained by comparing the number of "assessed polls" with the population in past census years. Other factors of similar character may be obtained by use of "school census" returns, the number of voters at recent elections, the number of water-service connections, the number of names in the directory, or Post Office or Police Department counts. None of these methods is, however, of great value in itself, but such methods may be utilized to confirm, or aid in forming, an estimate.

The future population may be predicted in a variety of ways which are more or less logical, and if employed with care, and if the data used in applying them are correct, the results will probably average as close to the truth as it is reasonable to expect such prophecies to be. The degree of accuracy is sufficient to enable a sewerage system to be designed with capacity enough to meet the requirements during the term of years for which it is planned, and yet not be of such great capacity that it throws an unwarranted financial burden on the community. These methods of predicting changes in population are:

1. By assuming that the rate of growth between recent census enumerations will remain constant for a considerable number of years.

2. By assuming that the rate of growth can be shown graphically by plotting a curve through the points representing the population of the city at different dates and then extending this curve into future years.

3. By assuming that the rate of growth will show a uniform arithmetical increase from one census year to another.

4. By assuming a steady decrease in the percentage rate of increase as the city grows larger and older.

Assumption of Uniform Rate of Growth.—A prediction of the increase in population, based on the assumption that the rate of growth between

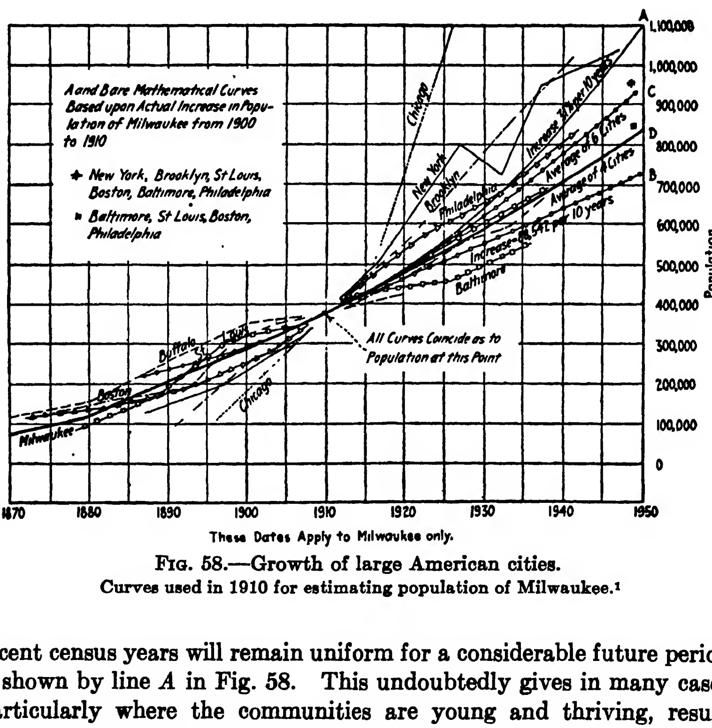


FIG. 58.—Growth of large American cities.
Curves used in 1910 for estimating population of Milwaukee.[1]

recent census years will remain uniform for a considerable future period, is shown by line A in Fig. 58. This undoubtedly gives in many cases, particularly where the communities are young and thriving, results which are too large, as indicated by the records of urban development. In view of this fact, the approval of the method contained in some of the early treatises on sewerage is an indication of the slight basis of fact on which the plans made then rested. For example, in Baldwin Latham's "Sanitary Engineering,"[2] the following advice is given:

[1] Population of Milwaukee in 1920 was 457,147.
[2] Edition of 1878.

The mode usually adopted in approximating the future population is to ascertain what has been the prospective rate of increase for a number of years back, and by making the same, or, in some cases, a greater allowance for increase in the future, so to calculate what is likely to be the probable population in years to come. In some districts this mode of estimating the population has been shown to be liable to error, as there are districts, such as manufacturing or suburban districts, located near large centers of population, which are liable to rapid rates of increase, and, in some cases, the population of particular manufacturing and mining districts has been found to decline.

This method was a favorite one in Germany down to about 1890, when it was discovered that many of the large cities which had increased uniformly from year to year from 1870 to about 1887 or 1888, had suddenly begun to grow at a much more rapid rate. Munich, Leipzig, and Cologne showed this change in an astonishing way. Until this rejuvenation took place, it was customary to predict the growth of German cities by the formula, $P = p[1 + (f/100)]^n$ where P is the population after n years have elapsed, p is the present population, and f is the annual percentage of increase in the population which has been observed. Practically, the growth of many of these cities could be satisfactorily represented by straight lines down to 1887. The growth of the population of the London metropolitan district from 1841 to 1891 was about 20 per cent every decade, whereas the decennial rate of growth in Berlin and its suburbs was more rapid and, as is to be expected in a place of such rapid development in population, industries and commerce, the rate of increase has been erratic, like that of many thriving American cities. The method of estimating population by a uniform rate of increase is apparently most reliable in the case of large and old cities not subject to periods of great commercial or industrial activity.

Graphical Method of Estimating Future Population.—The information furnished by diagrams of the past growth of cities is instructive, but an attempt to predict the future growth of a city from its past development, by extending the curve of that development, is likely to give misleading results, as will be shown later. Diagrams have a useful place in the study of changes in population, but they are not a substitute for an investigation of the various influences which have affected the city's growth in the past and may affect it in the future.

Arithmetical Increase in Population.—The method of predicting future population which is carried out by assuming that the increase from decade to decade is an arithmetical rather than geometrical progression gives the straight line shown in Fig. 58, line B. An instance of the use of this method was in the preparation of the estimate of the population of the Borough of Manhattan made by the Board of Water Supply of New York City. According to this assumption, the arithmetical increase will become nil when the population reaches 3,000,000, the

entire subsequent growth of the city taking place in the other boroughs. Dr. Walter Laidlaw estimated in 1908 that New York's population would increase in an arithmetical rather than geometrical progression, basing this conclusion on the relative growth of New York and the whole country, the probable distribution of future immigrants, and an increasing westward trend of the country's inhabitants.

Decrease in Percentage Rate of Growth as Cities Increase in Size.—As a general rule it is found that the larger the city becomes, the smaller

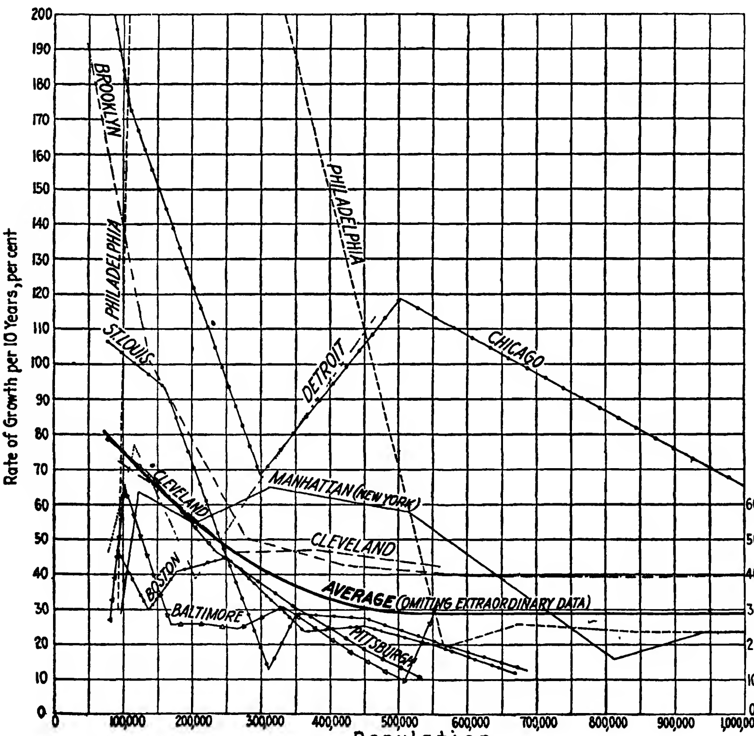


FIG. 59.—Relation of rate of growth of population to total population in American cities.

will be the percentage rate of growth from year to year. In Fig. 59 there are shown diagrams of the rates of growth of ten of the large cities of this country, in which the percentage growth in 10 years is plotted as ordinate against the population at the beginning of the decade as abscissa. The general reduction in rate of growth as the cities increase in size is distinctly marked. From this diagram the figures in Table 40

TABLE 40.—RATE OF GROWTH OF CITIES AT VARIOUS STAGES OF GROWTH; PERCENTAGE FOR 10-YEAR PERIODS

City	Population at beginning of decade									
	100,000	200,000	300,000	400,000	500,000	600,000	700,000	800,000	900,000	1,000,000
Manhattan Borough, New York	33	55	64	62	58	46	31	18	21	24
Brooklyn Borough, New York	140	79	49	44	41	40	40	39	39	40
Chicago	(184)	(123)	(69)	(93)	(118)	(108)	(97)	(87)	(76)	(65)
Philadelphia	107	(305)	(226)	(149)	(71)	21	25	24	24	24
Detroit	63	41	67	95						
Cleveland	71	57	46	46	44					
St. Louis	103	72	19	28	24	18				
Boston	44	42	36	24	23	17				
Baltimore	61	26	27	22	11					
Pittsburgh	75	55	37	25	13					
Average (omitting figures in parentheses)	77	53	43	43	31	28	32	27	28	29

TABLE 41.—RATE OF GROWTH OF AMERICAN CITIES, BY GEOGRAPHICAL DIVISIONS, 1900–1910 AND 1910–1920

Division	Cities over 100,000 in 1910		Cities over 100,000 in 1920		Cities of 25,000 to 100,000 in 1910		Cities of 25,000 to 100,000 in 1920	
	Number in 1910	Increase 1900–1910, per cent	Number in 1920	Increase 1910–1920, per cent	Number in 1910	Increase 1900–1910, per cent	Number in 1920	Increase 1910–1920, per cent
New England	8	21.2	11	15.5	34	29.0	35	20.2
Middle Atlantic	11	30.8	15	17 6	44	34.0	48	24.3
East North Central	10	32.3	12	36.9	38	37.8	61	43.5
West North Central	5	-30.4	7	20.0	17	25.2	14	23.2
South Atlantic	4	20.3	6	33.0	16	37.9	25	43.5
East South Central	4	34.6	4	15.8	7	21.9	7	22.6
West South Central	1	18.1	5	38.6	12	92.2	12	65.2
Mountain	1	51.9	2	22 4	5	54.5	5	16.9
Pacific	6	97.3	6	37 0	6	108.3	12	47.5
United States	50	32.8	68	24.9	179	37.9	219	33.0

	Cities of 2,500 to 25,000 in 1910		Cities of 2,500 to 25,000 in 1920		Territory rural in 1910		Territory rural in 1920	
	Number in 1910	Increase 1900–1910, per cent	Number in 1920	Increase 1910–1920, per cent	Number in 1910	Increase 1900–1910, per cent	Number in 1920	Increase 1910–1920, per cent
New England	320	16.7	246	14 7	..	−0.5	..	0.9
Middle Atlantic	444	39.7	541	21 0	..	8 7	..	4.7
East North Central	474	26.0	513	21.6	..	0.0	..	−0.6
West North Central	260	27.5	301	18.1	..	6.0	..	1.7
South Atlantic	190	42.7	242	28.3	.	12.3	..	7.8
East South Central	115	36.1	158	20 5	..	7.5	..	2.4
West South Central	177	80.7	243	35 8	..	27.1	..	8.7
Mountain	91	76.4	110	27.7	.	53.4	..	28.0
Pacific	103	109.3	147	41.4	.	46.4	..	22.6
United States	2,173	36.1	2,500	23.0	..	11.2	.	5.4

for rates of growth corresponding to populations of 100,000, 200,000, etc. have been taken, and these latter figures (omitting abnormal rates enclosed in parentheses) have been averaged at the bottom of the table. The curve representing these averages, after smoothing the irregularities, is shown by the heavy line in Fig. 59. All of the figures for Chicago and a portion of those for Philadelphia have been omitted as representing conditions so abnormal that they cannot fairly be included, even in making up averages.

These statistics show a gradual decrease in rate of population growth, from approximately 75 per cent in 10 years for cities of 100,000 population at the beginning of the decade, to approximately 29 per cent in 10 years at a population of 600,000, and a substantially uniform percentage rate thereafter. They are, however, to be taken only as representative of general tendencies and should not be applied directly in individual cases.

That rates of growth vary in different parts of the country and at different dates as well as with communities of different sizes, is distinctly shown in Table 41, based upon statistics published in the reports of the U. S. Census Bureau.

Decrease in Percentage Rate of Growth with Age.—In addition to the tendency toward the reduced rate of growth as cities grow larger, there is also a marked tendency toward a decreased growth as the nation grows older. In other words, the rate of growth, as a rule, for cities of similar size, was less between 1910 and 1920 than between 1870 and 1880, as shown by Table 42. This is also true of the population of the entire country, especially during the last half century, as shown by Table 43.

TABLE 42.—RATE OF GROWTH OF CITIES FROM DECADE TO DECADE

Decade	Cities between 100,000 and 200,000		Cities between 200,000 and 400,000	
	Number	Rate of increase, per cent	Number	Rate of increase, per cent
1840–1850	5	50.5		
1850–1860	5	63.2	2	80
1860–1870	6	52.4	6	60
1870–1880	10	49.5	6	26.5
1880–1890	12	76.8	9	40.8
1890–1900	19	33.4	13	25.9
1900–1910	22	38.8	17	42.2
1910–1920	35	27.9	17	27.1

Making the corrections suggested by the Census Bureau for the population of 1870, it appears that the rate of growth of the country per decade has decreased in 120 years from about 35 to 15 per cent, although the actual growth in numbers during this time has increased from decade to decade.

TABLE 43.—POPULATION AND RATE OF GROWTH OF UNITED STATES

Date	Population	Growth during decade	
		Numerical	Per cent
1790	3,929,214		
1800	5,308,483	1,379,269	35.1
1810	7,239,881	1,931,398	36.4
1820	9,638,453	2,398,572	33.1
1830	12,866,020	3,227,567	33.5
1840	17,069,453	4,203,433	32.7
1850	23,191,876	6,122,423	35.9
1860	31,443,321	8,251,445	35.6
1870	39,818,449[1]	8,375,128	26.6[1]
1880	50,155,783	10,337,334	26.0
1890	62,947,714	12,791,931	25.5
1900	75,994,575	13,046,861	20.7
1910	91,972,266	15,977,691	21.0
1920	105,710,620	13,738,354	14.9

[1] Census reports claim a deficiency in enumeration of Southern states for 1870. The Census Bureau gives estimated population and percentage as stated. The actual population as returned for 1870 was 38,558,371.

Methods of Estimating Future Growth.—Probably the best result to be derived mathematically may be obtained by assuming, in the light of the statements previously given, a decreasing rate of growth as time goes on, taking into consideration the size of the city at the end of each decade. Such an estimate is shown in Fig. 60. One of the most frequently employed and useful methods is to base the prediction on the experience of other cities which have already reached and passed the present population of the city under consideration. This is done, as shown in Fig. 58, by arranging the lines indicating the change in population of different cities so that when they have reached the present population of the city under consideration, they all pass through the same point. In this way their behavior, after passing this population, may be directly compared. This method may give results somewhat too high, as comparison is made with the past growth of cities, and, as already

pointed out, there is a tendency, as time goes on, for the rate of increase to become somewhat smaller.

It is usually desirable in such studies to investigate the growth of other cities in the vicinity at the same time as the growth of the city under special consideration, for the information thus obtained will generally reveal any local peculiarities in the increase of population.

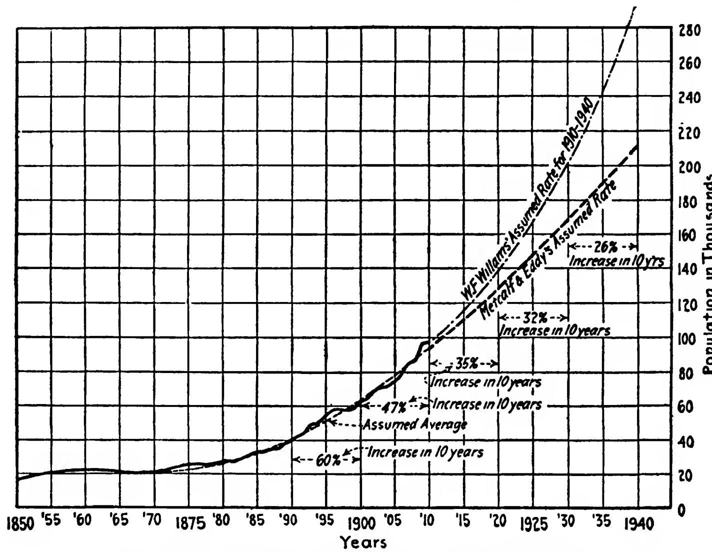


FIG. 60.—Growth of population of New Bedford.[1]

Increase in Area.—In estimating the probable quantity of sewage to be provided for by main and intercepting sewers, it is important to take into account the probable increase in the area served by sewers and, in many cases, the probable increase in area resulting from extension of the city limits. Such enlargements of area may cause large and sudden increases in population, which, if not anticipated, may cause the overtaxing of main sewers during the period for which they were intended to be adequate. Furthermore, such increases in area often require long extensions in main sewers and may result in greatly increased quantities of ground water made tributary to the mains and intercepters. Where the community is served by combined sewers, there is also the probability that for considerable periods in the future, or until the population becomes quite dense, brooks may be turned into the trunk sewers, thus adding materially to the nominal dry-weather flow of sewage. It is also of vital importance to consider where the estimated increase in population may occur in order that the lower sections of the main sewer

[1] Actual population in 1920 was 121,217; and in 1925, 119,539.

may be placed at elevations from which it will be possible to make extensions into new territory that may become populated within the period for which it is designed.

An illustration of increase in area by annexation is furnished by the growth of Cincinnati, shown in Table 44, compiled from data published in a general report on the disposal of the sewage of Cincinnati submitted in 1913 by H. M. Waite, H. S. Morse, and Harrison P. Eddy.

TABLE 44.—ANNEXATIONS TO THE CITY OF CINCINNATI, 1819–1913

Date of annexation	Area annexed, square miles	Total area, square miles	Date of annexation	Area annexed, square miles	Total area, square miles
1819[1]	.....	3.00	1903	5.13	41.96
1849	2.93	5.93	1904	0.47	42.43
1850	0.23	6.16	1905	0.59	43.02
1855	0.77	6.93	1907	0.48	43.50
1870	12.12	19.05	1909	6.03	49.53
1873	4.48	23.53	1910	0.73	50.26
1889	0.20	23.73	1911	16 03	66.29
1896	11.38	35.11	1912	2.45	68.74
1898	0.16	35.27	1913	1 11	69.85
1902	1.56	36.83	1920	....	71.14[2]

[1] Original city of Cincinnati; incorporated as a town in 1802, as a city in 1819.
[2] Actual area in 1920 as per census report.

There is a marked tendency at present, doubtless encouraged by constantly improving transportation facilities, for the inhabitants of cities to move into suburban districts. This condition tends toward a lower density of population, although it is more effective in reducing the probable increase in density than in diminishing existing density. As the suburban areas become more thickly populated, the improvements of the cities are desired there and are ultimately demanded. To secure these, it often becomes necessary for suburban districts to be annexed to the city, thus extending the city limits. It is reasonable, therefore, to expect a city to increase in area as well as population. In making studies of the future sewerage needs of Fort Wayne, for instance, the authors estimated that the area would grow from 8.6 square miles in 1910 to 17.3 square miles in 1950. In a number of places, municipal boundaries have been ignored in water supply and sewerage undertakings, as at Boston, Mass., and several sections of the territory about New York.

This tendency of large cities to develop by the absorption of adjoining communities, or by the delegation of full authority over certain classes

of public works to commissions acting for the entire district served, has led the Bureau of the Census to pay special attention to metropolitan districts, because

> . . . in some cases, the municipal boundaries give only an inadequate idea of the population grouped about one urban center; in the case of many cities, there are suburban districts with a dense population outside the city limits, which, in a certain sense, are as truly a part of the city as the districts which are under the municipal government.

The 1910 census showed that in 25 such metropolitan districts, the average percentage of increase in the cities during the last decade had been 33.2 per cent and in the suburbs 43 per cent. Corresponding figures from the 1920 census for 29 such districts were 25.1 and 32.7 per cent. But these average figures are extremely misleading when used as a guide to the development of the smaller metropolitan districts, because they are greatly influenced by the growth of districts with more than 500,000 population, and the location and age of a city are of much influence on the development of its suburbs as well as of itself. For example, Providence and Detroit had about the same population in 1900, but the development of the Providence metropolitan district in the following decade was only 29.4 per cent while that of the Detroit district was 57.1 per cent. Furthermore, the development of the Providence suburbs was more rapid than that of the city, whereas the development of Detroit was almost wholly in the city proper. In the 1910–1920 decade the differences had become even more marked; the entire Providence district increased only 12.2 per cent, while the Detroit district increased 126.6 per cent, and the rate of growth of the suburbs had passed that of the city proper, the percentages being 254.9 and 113.3, respectively.

Density of Population.—The average density of population varies greatly in different cities, as is shown in Table 45. It should be noted that the greatest average density is found in cities of small area; while the largest cities have moderate average densities on account of large areas of parks and of sparsely populated districts.

In designing sewers for a community it becomes necessary to estimate the probable distribution of population within the city. This is largely a matter of conjecture, except in the sections of greatest age, as the density may vary from 2 or even less per acre in outlying districts to 150 or more per acre in the most densely settled parts of some large cities. The New York Metropolitan Sewerage Commission estimated that the future density of population in the part of Manhattan which drains into the Hudson River from the Battery to the Harlem River will be 306 persons per acre; that of the part of the Borough of the Bronx draining into the Harlem River will be 239 persons per acre, and that

TABLE 45.—AVERAGE DENSITY OF POPULATION FOR THE 52 CITIES OF OVER 50,000 POPULATION IN WHICH THE DENSITY IS GREATEST

Computed from data in Census Bureau's "Financial Statistics of Cities" for 1910 and 1921

City	Density, persons per acre			Population, 1920	Area land surface within city limits, acres, 1920
	1920	1910	1900		
1. Hoboken, N. J.	82	85	71	68,166	830
2. Somerville, Mass.	37	30	24	93,091	2,518
3. Jersey City, N. J.	36	32	25	298,103	8,320
4. Passaic, N. J.	32	26	13	63,841	2,002
5. New York, N. Y.	30	26	19	5,620,048	191,360
6. Bayonne, N. J.	30	22	13	76,754	2,544
7. Milwaukee, Wis.	28	25	22	457,147	16,290
8. Newark, N. J.	28	23	21	414,524	14,912
9. Boston, Mass.	27	27	23	748,060	27,635
10. Trenton, N. J.	27	22	16	119,289	4,490
11. Cambridge, Mass.	27	26	23	109,694	4,002
12. Paterson, N. J.	26	24	20	135,875	5,157
13. Altoona, Pa.	26	25	23	60,331	2,317
14. Wilmington, Del.	25	22	19	110,168	4,495
15. Norfolk, Va.	24	19	16	115,777	4,800
16. Camden, N. J.	24	21	17	116,309	4,915
17. Charleston, S. C.	24	24	23	67,957	2,874
18. Pittsburgh, Pa.	23	20	13	588,343	25,530
19. Chicago, Ill	22	19	14	2,701,705	123,383
20. Philadelphia, Pa.	22	19	16	1,823,779	81,920
21. Cleveland, Ohio	22	19	17	796,841	36,089
22. Lawrence, Mass.	22	21	15	94,270	4,317
23. Wilkes-Barre, Pa.	22	21	16	73,833	3,326
24. Detroit, Mich.	21	18	16	1,044,860	49,839
25. Providence, R. I.	21	19	15	237,595	11,388
26. Lancaster, Pa.	21	19	16	53,150	2,530
27. St. Louis, Mo.	20	17	15	778,897	39,040
28. Buffalo, N. Y.	20	17	14	506,775	24,894
29. Harrisburg, Pa.	20	19	17	75,917	3,766
30. East Orange, N. J.	20	14	9	50,710	2,516
31. San Francisco, Calif.	19	14	12	506,676	26,880
32. Savannah, Ga.	19	16	18	83,252	4,473
33. Johnstown, Pa.	19	20	15	67,327	3,488
34. Chester, Pa	19	13	11	58,030	3,020
35. Reading, Pa.	18	24	20	107,784	6,091
36. Schenectady, N. Y.	18	15	11	88,723	5,019
37. Portsmouth, Va.	17	21	19	54,387	3,136
38. Columbus, Ohio	16	14	12	237,031	14,427
39. Louisville, Ky.	16	17	16	234,891	14,349
40. Dayton, Ohio	15	12	13	152,559	10,107
41. Bridgeport, Conn.	15	13	9	143,555	9,370
42. Lynn, Mass.	15	13	10	99,148	6,705
43. Elizabeth, N. J.	15	12	9	95,783	6,191
44. Evansville, Ind.	15	16	14	85,264	5,577
45. Racine, Wis.	15	13	10	58,593	3,858
46. Covington, Ky.	15	17	24	57,121	3,837
47. Baltimore, Md.	14	29	26	733,826	50,560
48. Rochester, N. Y.	14	17	14	295,750	20,566
49. Syracuse, N. Y.	14	12	10	171,717	11,849
50. New Haven, Conn.	14	12	9	162,537	11,460
51. Hartford, Conn.	14	9	7	138,036	10,162
52. Tulsa, Okla.	14	..	..	72,075	5,002

Note.—Some cities show a decrease in density between censuses, due to the annexation of large areas of adjacent territory.

of the district draining into the Lower East River will be 198 persons per acre.[1] The probable lowest density in any district, 8 per acre, will be in the territory draining into the Upper East River. These figures were obtained by taking the probable population of Manhattan and Brooklyn as of 1,960; Queens as of 1,950; and the Bronx as of 1,940. Furthermore, the character of the various parts of a city changes. A residential section of the present decade may become the commercial or manufacturing district of the next decade, or the change may be in the type of residential development from a section containing the homes of a people of considerable means to a congested tenement district. These influences may result in increasing the density, causing it to remain nearly stationary, or even decreasing it in some cases.

Figures for average density of population may be and usually are of little significance, since all cities which cover extensive areas include sections in which the population is sparse, and sometimes considerable areas of parks, cemeteries, railroad yards, etc. The density of population in small cities which are concentrated in small areas may often be much greater. For instance, in 1920, West Hoboken and Union, N. J., showed densities of 76 per acre; West New York, N. J., 49; Braddock, Pa., 63; Homestead, Pa., 60; and Hamtramck, Mich., 42 per acre.

Still more significant in estimating the population to be served by a sewerage system are the densities of population in wards of cities.

A study of the density of population in the different wards of Boston as shown by four censuses at 5-year intervals, and of the changes in density during the intervening 15 years, reveals some facts which may aid in predicting the growth of other cities of similar character. The statistics are given in Table 46. The city may be divided for this purpose into outlying sparsely settled regions, good residential districts, fairly densely populated business and commercial districts, and cheap tenement districts. The increase in density of the sparsely settled districts, wards 23, 24 and 25, was very slow, amounting to only 1 or 2 persons per acre per 10 years. When, however, such districts became fairly well built up and desirable residential sections with densities of about 20 to 25 per acre, the increase became rapid, amounting for example in wards 20 to 23, to from 5 to 13 persons per acre per 10 years. The lodging-house districts and business sections or those in a transitory stage remained nearly uniform or even decreased in density under certain conditions. The sections with the tenement houses of lowest rental, as ward 8, appear to be increasing rapidly in spite of a density already very great. In fact, the greatest increase in the whole city in the 15 years has taken place in those sections, and it appears to be hazardous to assume that the density in such districts, because it is already high, will

[1] A recent newspaper article (1925) states that the present population in the lower east side of Manhattan Borough is 219,256 in an area of 429 acres—a density of 511 per acre.

not go on increasing. Where a district is in a transitory stage, as between a place for business and a place for residence, its ultimate course may largely affect the density. If it becomes commercial, the density may not change greatly or may decrease, whereas if it changes into a cheap tenement region, the density may go on increasing to a very high figure.

TABLE 46.—GROWTH IN POPULATION OF THE WARDS OF THE CITY OF BOSTON

Ward	Area land in acres	1895		1900		1905		1910	
		Popul. per acre	Per cent of popul.	Popul. per acre	Per cent of popul.	Popul. per acre	Per cent of popul.	Popul. per acre	Per cent of popul.
1	1,188	17.7	4.23	19.2	4.07	21.4	4.27	24.9	4.43
2	357	60.5	4.34	64.2	4.09	72.6	4.35	80.7	4.30
3	332	42.0	2.81	43.9	2.60	44.7	2.49	46.2	2 29
4	301	44.4	2.69	44.0	2.36	41.5	2.10	44.1	1.98
5	207	62.7	2.61	62.0	2.29	61.7	2.12	61.9	1.91
6	293	95.1	5.61	104.3	5.45	102.3	5.04	122.0	5.33
7	394	43.0	3.42	37.5	2.64	39.5	2.62	37.9	2.22
8	171	135.0	4.65	168.5	5.14	180.4	5.17	190.0	4.84
9	186	124.5	4.66	132.0	4.38	118.9	3.72	141.5	3.94
10	394	57.2	4.54	56.2	3.95	60.5	4.00	64.3	3.78
11	663	30.0	4.01	29.1	3.44	33.7	3.75	41.4	4.09
12	235	92.0	4.35	100.6	4.21	92.5	3.65	103.4	3.62
13	611	40.7	5.01	37.4	4.07	35.4	3.64	35.3	3.22
14	405	47.4	3.86	53.0	3.82	54.7	3.72	58.2	3.52
15	277	67.2	3.75	71.1	3.51	73.3	3.41	76.6	3.16
16	564	28.9	3.28	35.5	3.57	38.9	3.68	45.6	3.82
17	460	45.8	4.25	54.4	4.46	52.8	4.08	57.4	3.94
18	220	98.6	4.36	101.9	3.99	100.6	3.72	103.3	3.39
19	760	29.4	4.50	35.7	4.85	38.4	4.91	41.7	4.73
20	1,716	12.6	4.33	19.0	5.80	24.4	7.02	32.5	8.31
21	640	30.1	3.88	37.3	4.26	41.5	4.46	50.5	4.55
22	760	29.3	4.49	33 7	4.57	36.5	4.66	38.1	4.47
23	7,617	2.4	3.68	3.1	4.21	3.5	4.44	4.0	4.57
24	3,252	5.6	3.67	8.3	4.83	9.7	5.32	11.6	5.63
25	2,740	5.5	3.02	7.0	3.44	8.0	3.66	9.7	3.96
Av. density		20.0		22.7		24.1		27.1	

Changes in ward boundaries between 1910 and 1915 make the figures of latter censuses incomparable with those in the above table.

TABLE 47.—POPULATION OF CHICAGO BY WARDS

Ward	Area, acres[1]	Population		Per cent change	Density	
		1900	1910		1900	1910
1	1,440	43,764	29,528	−33.0	30.4	20.5
2	800	44,583	42,801	− 4.0	55.6	53.5
3	960	44,425	46,135	4.0	46.3	48.1
4	960	49,058	49,650	1.0	51.1	51.7
5	2,240	48,206	57,131	19.0	21.5	25.5
6	1,600	57,831	75,121	30.0	36.1	47.0
7	4,160	55,074	90,423	64.0	13.2	21.7
8	13,624	49,493	65,810	33.0	3.6	4.8
9	640	45,984	44,801	− 3.0	71.8	70.0
10	640	47,525	51,707	9.0	74.3	80.8
11	1,120	57,601	57,664	0.1	51.4	51.5
12	2,880	50,246	91,521	82.0	17.4	31.8
13	1,600	43,266	58,721	36.0	27.1	36.7
14	1,280	49,299	52,770	7.0	38.5	41.2
15	1,120	49,178	60,438	23.0	43.9	53.9
16	800	58,158	65,223	12.0	72.8	81.5
17	720	66,084	70,099	6.0	91.9	97.4
18	640	31,404	26,137	−17.0	49.1	40.8
19	640	52,024	58,023	12.0	81.3	90.7
20	800	49,271	61,708	25.0	61.6	77.1
21	960	50,283	47,906	− 5.0	52.4	49.9
22	960	52,523	49,324	− 6.0	54.7	51.4
23	800	45,601	44,320	− 3.0	57.0	55.4
24	1,120	43,465	52,428	21.0	38.8	46.8
25	4,160	54,588	99,696	83.0	13.1	24.0
26	4,640	43,354	74,793	72.0	9.3	16.1
27	20,480	44,290	112,793	156.0	2.1	5.5
28	1,760	55,605	68,183	23.0	31.6	38.7
29	6,400	51,243	81,985	60.0	8.0	12.8
30	1,280	52,757	51,308	− 3.0	41.2	40.1
31	11,200	50,954	78,571	54.0	4.5	7.0
32	8,480	40,211	70,408	75.0	4.7	8.3
33	12,944	37,100	70,841	91.0	2.9	5.5
34	3,200	26,611	67,769	155.0	8.3	21.2
35	4,960	28,086	59,547	112.0	5.7	12.0
Total......	122,008	1,698,575	2,185,283	28.6	13.9	17.9

[1] Includes water surface.

Similar tendencies in Chicago are indicated in a report on sewage disposal in the Chicago Sanitary District, by G. M. Wisner. The average density of the four most densely populated wards increased from 76.2 to 86.2 in 10 years (Table 47). The tendency was for the density of business sections either to stand still or to decrease somewhat. It should be borne in mind, however, that this refers to resident population, and that the number of persons present in the district during the business hours is probably increasing at a rapid rate.

Between 1910 and 1920 there were radical changes in ward boundaries and also in the numbering of wards, so that figures for population and density in wards in 1920 are not comparable with those in the table. In 1925 a new division of the city was made, this division being into 50 wards substantially equal in population, but varying widely in area. The density of population in these wards ranged from 4 to 86 per acre.

Effect of Zoning.—The most direct effect that the engineer realizes in undertaking to design a sewerage system, in a community which has been zoned and which has an effectively administered zoning ordinance, is the relative certainty and economy with which he can adapt the sewers to the service they will be called on to perform . . . The perfection of sewer performance must be proportioned to property values. In zoned areas, the property values are relatively easy to determine and also relatively stable, so that the engineer has a much more reliable guide to judgment than he has when the future character and, therefore, the future value of the property is uncertain.[1]

Hansen has shown that, with a typical block 330 by 660 ft., and lots 50 ft. wide, the densities of population and percentages of impervious area would be about as follows:

Character of district	Development	Density of population, persons per acre	Impervious surface, per cent
Dense residential......	Two-family houses and six-family apartment buildings	55	60
Medium residential....	Mostly single-family houses	35	43
Light residential.......	Single houses only, some on double lots	15	31
Mercantile...........	....	14	100
Light commercial......	....	30	80
Industrial...........	....	10	60

[1] HANSEN, PAUL, "The Relation of Zoning to the Design of Drainage and Sewerage Systems," *Trans. Am. Soc. C. E.*, 1925; **88**, 680.

Zoning may also be advantageous in preventing low-lying land from being occupied in such a way as to require drainage or sewerage, or at least to restrict the uses of such districts so as to avoid the worst difficulties attendant upon poor natural drainage.

Accuracy of Population Estimates.—It appears that, in the majority of cases, forecasts of population made by the methods described above indicate somewhat larger populations than those actually found by census counts. From a comparison of some 40 population estimates with census figures, a number of them covering periods of 35 years since the date of the last census preceding the estimate, it appears that on the average the estimated population has exceeded the actual population by about 1 per cent for each year which has elapsed.

As a rule, curves of actual population growth are not regular but fluctuate somewhat widely. Curves of predicted population are always smooth curves. Therefore, it is not to be expected that actual populations for any particular place should correspond closely to the estimated population for each of several census periods. The deviation is likely to be more marked with smaller than with larger cities, since the former are more susceptible to the effect of various conditions affecting the rate of growth. Therefore, it is possible that an estimate which may ultimately prove to be a good representation of average conditions may depart widely from census figures at each of several census periods. Differences amounting to 20 per cent or even more do not necessarily indicate that the estimate is diverging materially from the general trend.

A series of estimates of the population of 27 cities and towns within 10 miles of Boston, made by Frederic P. Stearns for the Report on a Metropolitan Water Supply (1895) is unusually complete and affords a large amount of data in this connection. Because of his long experience and personal familiarity with the communities under consideration, Stearns was peculiarly well qualified to make such estimates. Massachusetts takes a state census between the Federal censuses, so that population counts are available at 5-year intervals. Stearns' estimates were made before the 1895 counts had been taken; consequently, the latest census used was that of 1890. There have been 7 census counts since the estimates were made. In all, 186 comparisons between estimated and actual populations are possible. These show 27 cases in which the population was underestimated and 156 in which it was overestimated, while in 3 cases the estimates agreed with the actual populations. The averages of the percentage relations vary from an excess of 2 per cent after 5 years to one of 44 per cent after 35 years. The range of differences varied from 10 per cent under estimate to 30 per cent over estimate after 5 years, and from 17 per cent under estimate to 90 per cent over estimate after 35 years.

Comparison of the figures for individual places shows clearly the effect of local circumstances, such as a serious fire checking development, unexpected growth due to the provision of rapid-transit facilities to certain suburban districts, or abnormal and unanticipated industrial development.

In so far as conclusions may be drawn from these figures, it appears that forecasts of population based upon experience of the past are likely to prove somewhat too high in about 85 per cent of the cases, and too low in the remainder; that in extreme cases the divergence may amount to as much as 80 or 90 per cent over the true population in a period of 35 or 40 years, or it may fall short of the true population by 20 per cent in the same period of time.

These figures are believed to justify the statement that forecasts of population prepared by the methods described above provide as reasonable estimates of future conditions as it is feasible to make. This is especially true for short-time predictions of, say, 15 to 20 years. The estimates are more likely to be too high than too low, which is desirable.

RELATION BETWEEN WATER CONSUMPTION AND SEWAGE

Proportion of Water Supply Reaching Sewers.—It is natural to think of sewage as consisting of the municipal water supply defiled by the wastes of the community, in which case the quantity of water consumed would be an accurate measure of the quantity of sewage produced. This impression, however, is incorrect, as only a portion of the municipal water supply reaches the sewers, and this may constitute less than half of the sewage. Much water from other sources also goes into the sewers.

A considerable part of the water supply used by railroads, by manufacturing establishments and power plants, in street and lawn sprinkling, for extinguishing fires, and by consumers not connected with the sewers, fails to reach the sewers, and there is usually considerable leakage from

TABLE 48.—ESTIMATED QUANTITY OF WATER SUPPLIED AND NOT REACHING THE SEWERS, IN MILWAUKEE, 1911

	Gallons per capita daily
Steam railroads	5
Manufacturing and mechanical purposes	5
Street sprinkling	5
Lawn sprinkling	2½
Consumers not connected with sewers	7½
Leakage from mains and services	15[1]
Total	40

[1] The leakage probably greatly exceeds this in many cities.

mains and service pipes. The Milwaukee Sewage Disposal Commission estimated in 1911 that the quantity of water supply for the several purposes listed in Table 48 never reached the sewers. This is a total of 40 gal., or 38 per cent of the supply at the time, which was 105 gal. per capita daily.

TABLE 49.—RATIO OF SEWAGE FLOW TO CONSUMPTION OF WATER IN VARIOUS CITIES DURING SUCCESSIVE YEARS, PER CENT

	Mass. North Metropolitan Sewerage District	Worcester, Mass.	Brockton, Mass.	Quincy, Mass.	Providence, R. I.
1900	..	155	59		
1901	...	109	66		
1902	...	158	56		
1903	...	161	60	...	184
1904	121	124	56	130	178
1905	111	123	54	105	188
1906	119	159	73	117	165
1907	121	164	66	120	145
1908	111	169	69	123	150
1909	122	163	63	143	160
1910	119	136	63	143	120
1911	112	143	65	Fitchburg, Mass.	155
1912	115	142	72	...	162
1913	126	154	89	...	173
1914	127	147	63	...	195
1915	140	130	84	...	168
1916	145	145	85	72	186
1917	137	126	83	82	182
1918	123	142	80	64	154
1919	143	148	110	79	171
1920	143	147	94	71	102
1921	139	141	107	59	154
1922	141	143	114	81	149
1923	136	137	98	77	141
1924	123	135	96	85	147
1925	122	137	98	78	176
1926	123	144	94	80	182

It is probably true that in many places some of the leakage from mains and services ultimately finds its way into the sewers by infiltration, but it is impossible to determine the proportion and it will vary greatly in different communities. In spite of the fact that all of the municipal

water supply does not reach the sewers, it is important to know its quantity and to use the data in forming an estimate of the quantity of sewage which will be produced, particularly during the dry season of the year. That the water supply is a very important item in the flow of sewage is indicated by Table 49. It will be seen that although the relation between the two varies widely in different cities, the relation is a fairly constant one in the same city from year to year.

Continuous gagings for one year (1926–1927) at Denver, Colo., showed monthly ratios of sewage to water consumption ranging from 57 to 97 per cent and averaging 76 per cent.[1]

Water Consumption in Cities.—The consumption of water in American cities, particularly the different classes of consumption and the variations in the hourly, daily, weekly, and monthly rates at which water is used, is discussed in detail in a report by Metcalf, Gifford, and Sullivan[2] upon which much of the following discussion of the subject has been based.

Table 50 gives the approximate population served, the water consumption in gallons per day per capita, and the percentage of services metered, for 12 typical American cities, yearly from 1890 (or such later date as the figures are available) to 1926. It will be seen that in nearly all the cases reported, most of the services are now metered. In general, the introduction of meters has been accompanied or followed by a considerable reduction in water consumption. This has usually been followed later by a moderate increase in consumption, due in large measure to the greater facilities for using water provided by modern plumbing systems, and, as X. H. Goodnough has noted,[3] "When the amount of water he uses is measured, and he pays for just what he uses, the householder soon learns that its cost is insignificant as compared with its value." At the same time, the effect of leaks appears in the bill, so that leaks of any consequence in house plumbing will be repaired promptly.

Goodnough found[4] that, in 22 American cities, the annual increase in daily consumption per capita after about 75 per cent of the services were metered, has ranged from about 0.75 to 2.50 gal. and averaged about 1.42 gal.

The Milwaukee Sewage Disposal Commission, which studied this question carefully, said in 1910 that, taking into account the history of the Milwaukee water works, the industrial character of the city, the low water rate of 6 cts. per 1,000 gal., as well as the availability of river and lake water, it was of the opinion that an increase of 5 gal. per day per capita per decade was a reasonable allowance to make for the next

[1] *Eng. News-Rec.*, 1928; **100**, 559.
[2] *Jour. New Eng. Water Works Assoc.*, 1913; **27**, 29.
[3] "Report on Water Supply Needs and Resources of Massachusetts," 1922, p. 79.
[4] *Loc. cit.*, pp. 65 and 80.

40 years. The figures in Table 50, however, show an increase of 24 gal. between 1910 and 1920, and an actual decrease from 1920 to 1925.

TABLE 50.—WATER CONSUMPTION AND METERING IN TYPICAL AMERICAN CITIES

Place	Item	1900	1905	1910	1915	1920	1925
Brockton, Mass	Population	....	55,700	66,000	74,588	79,729	83,548
	Consumption	....	36	36	38	43	50
	Per cent metered	....	90	99	100	100	98
New Bedford, Mass.	Population	62,500	75,000	96,652	110,000	131,350	146,800
	Consumption	101	95	79	70	78	65
	Per cent metered	15	23	48	96	96	93
Worcester, Mass.	Population	118,421	132,550	150,738	169,599	179,923	191,000
	Consumption	69	73	71	75	90	82
	Per cent metered	94	96	97	100	97	97
Providence, R. I.	Population	187,297	214,335	246,000	278,727	270,472	312,615
	Consumption	54	68	63	62	80	75
	Per cent metered	83	86	89	93	94	97
Hartford, Conn.	Population	...	91,000	105,500	138,000	170,000	188,000
	Consumption	...	69	70	67	71	86
	Per cent metered	...	99	99	98	97	96
Cleveland, Ohio	Population	397,200	462,000	604,073	765,000	925,283	1,098,466
	Consumption	169	131	102	102	152	148
	Per cent metered	6	70	98	99	98	99
Cincinnati, Ohio	Population	325,902	343,254	363,591	410,000	401,247	416,000
	Consumption	117	129	128	119	123	119
	Per cent metered	8	12	33	65	100	100
Indianapolis, Ind	Population	......	......	237,000	269,500	322,200	372,000
	Consumption	....	....	81	82	94	93
	Per cent metered			11	12	14	31
Milwaukee, Wis	Population	285,315	335,000	390,000	450,000	500,000	565,000
	Consumption	83	91	109	105	133	128
	Per cent metered	86	94	98	99	99	99
New Orleans, La.	Population	....	......	....	363,100	387,219	411,000
	Consumption	....	....	....	71	107	120
	Per cent metered	......	....	....	100	100	100
San Antonio, Tex.	Population	....	....	99,000	132,000	161,000	
	Consumption	......	.....	128	117	126	
	Per cent metered	......	....	18	31	43	
San Francisco, Cal.	Population	343,000	384,000	417,000	500,000	575,000	
	Consumption	74	91	85	85	65	
	Per cent metered	23	21	27	31	100	

Population served by water works.
Consumption in gallons per day per capita.
Percentage of services which are metered.

Fluctuations in Water Consumption.—While it is important to know the average quantity of water consumed, it is of still greater value to have data relating to the fluctuations in consumption, as a sewer must be designed to take the sewage when flowing at its maximum rate. The maximum rate of water consumption usually occurs during summer months when water is in demand for street and lawn sprinkling and the excess is not likely to reach the sewers, or in the winter when large

Table 51.—Records of Maximum Water Consumption for Massachusetts Cities and Towns, 1910

City or town	Population	Average daily consumption per person	Max. monthly consumption		Max. weekly consumption		Max. daily consumption	
			Gal. per person per day	Per cent of average for year	Gal. per person per day	Per cent of average for year	Gal. per person per day	Per cent of average for year
Abington and Rockland	12,383	45	63	137	70	151	90	197
Amesbury	9,894	44	50	114	51	116	68	155
Andover	7,301	86	99	115	...	...	162	189
Attleborough	16,215	54	62	115	63	116	91	169
Avon	2,013	36	55	153	72	200	102	283
Ayer	2,797	50	64	128	72	144	172	342
Beverly	18,650	91	146	160	191	210	224	246
Braintree	8,066	81	87	107	93	115	108	133
Bridgewater and E. Bridgewater	11,051	22	27	123	29	132	39	177
Brockton	56,878	39	45	115	55	141	69	177
Brookline	27,792	89	103	116	117	132	128	177
Cambridge	104,839	100	106	106	111	111	119	119
Canton	4,797	61	75	123	84	138	97	159
Danvers and Middleton	10,536	89	108	121	136	153	158	178
Dedham	9,284	129	153	119	163	128	182	141
Easton	5,139	24	28	117	38	158	63	263
Fall River	119,295	44	47	107	50	114	54	123
Foxborough	3,863	50	51	102	59	118	75	150
Framingham	12,948	48	58	121	65	137	86	179
Franklin	5,641	61	88	144	96	158	127	208
Gardner	14,699	44	49	111	55	125	108	246
Gloucester	24,398	55	96	156	114	207	130	237
Grafton	5,705	18	22	122	24	133	32	178
Hudson	6,743	49	60	123	65	133	...	
Ipswich	5,777	42	60	143	84	200	106	253
Lawrence	85,892	45	51	113	60	133	60	133
Lowell	106,294	51	57	112	66	129	75	147
Lynn and Saugus	97,383	72	79	110	87	121	108	150
Manchester	2,673	120	261	217	327	271	363	302
Mansfield	5,183	75	97	129	103	137	276	368
Marblehead	7,338	79	147	186	169	214	187	237
Marlborough	14,579	37	42	114	59	159	80	190
Maynard	6,390	36	39	108	47	130	60	167
Methuen	11,448	38	54	142	67	176	69	182
Middleborough	8,214	42	53	126	65	155	90	214
Milford and Hopedale	15,243	51	60	118	64	125	71	139
Montague and Erving	8,014	66	75	114	70	106	153	232
Nantucket	2,962	67	128	191	154	230	176	263
Natick	9,866	57	70	123	82	144	170	298
Needham	5,026	66	88	133	98	148	119	180
New Bedford	96,652	81	88	109	98	121	106	131
Newburyport	14,949	68	83	122	94	138	121	178
Newton	39,806	63	74	118	82	130	95	150
North Andover	5,529	40	53	132	64	160	78	195
North Attleborough	9,562	52	73	140	82	158	95	183
North Brookfield	3,075	66	81	123	112	170	213	319
Norwood	8,014	63	86	136	86	136	132	211
Orange	5,282	26	34	131	41	158	62	182
Peabody	15,721	168	198	118	182	108	270	161
Plymouth	12,141	103	131	127	140	136	171	166
Provincetown	4,369	38	69	182	77	203	93	245
Randolph and Holbrook	7,117	74	120	162	140	189	175	237
Reading	5,818	35	52	149	60	172	66	189
Rockport	4,211	72	148	205	196	272	212	295
Salem	43,697	90	101	112	103	114	133	148
Sharon	2,310	57	97	170	120	210	137	240
Stoughton	6,316	35	43	123	58	166	75	214
Taunton	34,259	63	70	111	74	118	87	138
Wakefield	11,404	61	85	139	107	175	127	208
Walpole	4,892	102	119	117	149	146	252	247
Waltham	27,834	88	95	108	98	111	108	123
Webster	11,509	38	49	129	53	139	72	189
Wellesley	5,413	61	68	111	79	129	112	184
Whitman	7,292	29	42	145	44	152	...	
Winchendon	5,678	30	35	117	40	133	45	150
Woburn	15,308	139	172	124	190	137	232	167
Worcester	145,986	74	85	115	...	...	103	139
Average	......	**68**	**81**	**128**	**93**	**147**	**123**	**198**

quantities are allowed to run to prevent freezing of pipes and fixtures, this excess usually finding its way into the sewers. In Table 51 have been compiled records of maximum water consumption for 67 Massachusetts cities and towns (1910) taken from report of Committee on Water Consumption Statistics and Records.[1] The average water consumption in the cities and towns included in this compilation was 63 gal. per capita per day. The average maximum monthly consumption, the maximum weekly consumption, and the maximum daily consumption were 128, 147, and 198 per cent of the average daily con-

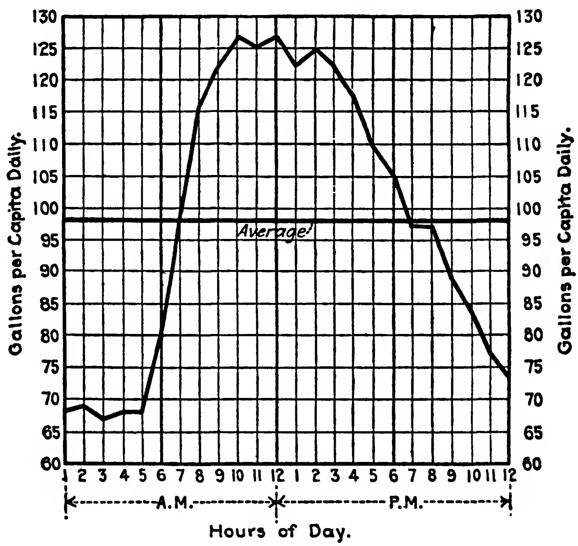


FIG. 61.—Hourly water consumption for average day in Holyoke in November 1905.

Estimated population supplied, 51,000.

sumption for the year, respectively. There were, however, instances in which the maximum rates greatly exceeded these averages. For example, in Manchester and Mansfield, Mass., the maximum daily consumption was 302 and 368 per cent of the average for the year, respectively. These high rates of flow, however, almost always occur at times when the usual proportion of the flow does not reach the sewers, as in the driest portion of the summer, or in winter when water from other sources may be at a minimum.

In addition to the fluctuations in flow already discussed, there is an important variation from hour to hour each day, as illustrated by Figs. 61 and 62 taken from the same report. It will be seen from Fig. 62

[1] *Jour. New Eng. Water Works Assoc.*, 1913; **27**, 29.

that the maximum peak flow during the week occurred on Monday, when the draft was about 135 per cent of the average for the day, and the minimum peak draft was on Sunday when it was 146 per cent of the average for the day, these rates being 139 per cent and 132 per cent, respectively, of the average rate of draft for the week.

The hourly fluctuation in rate of water consumption has a decided effect upon the rate of sewage flow, as discussed later in this chapter. It is not, however, entirely responsible for the fluctuation in the rate of flow of sewage, for in some places large quantities of ground water are pumped by industrial establishments and discharged into the sewers

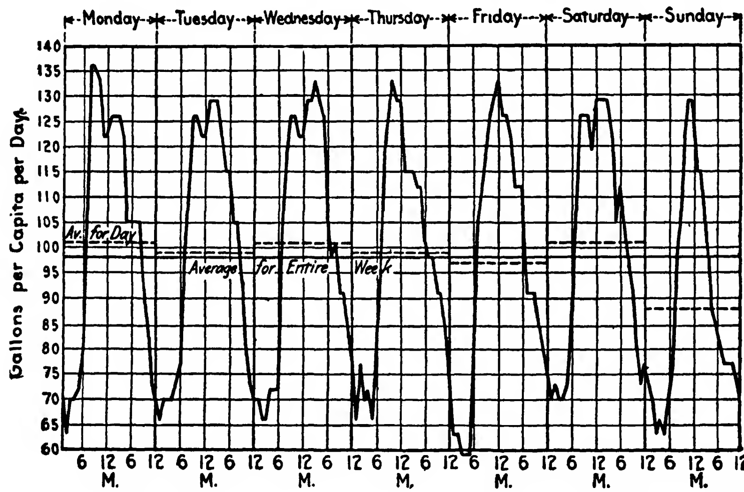


FIG. 62.—Fluctuations in water consumption in Holyoke during week ending November 17, 1905.

during the working hours of the day, thus tending to increase the peak flow beyond the amount resulting from the normal fluctuation in the draft on the municipal water supply.

Allowing for water drawn from private supplies, the peak rate of consumption during the day of maximum use may be taken at 150 per cent of the average draft upon the municipal supply for that day. This rate, however, will vary in different places.

If this peak rate is applied to the maximum draft for a single day, taken as 198 per cent of the average annual consumption, and it is assumed that the portion of the public water supply which finds its way into the sewers averages 50 gal. per day, we have a maximum rate of contribution from the public and private water supplies of about 150 gal. per capita daily ($50 \times 1.98 \times 1.50 = 148.5$). This will serve to

illustrate the theory of the yield of sewage based on water consumption, the figures for each case to be selected as local conditions may warrant.

Rate of Consumption in Different Parts of a City.—The consumption of water, and consequently the volume reaching the sewer, varies greatly in different districts of a city. The total volume of water delivered is made up of the requirements for public, domestic, and industrial uses and an amount which is usually termed "waste," although "unaccounted-for water" might be a better term. Water used for industrial purposes was found in 1904 in the Massachusetts Metropolitan Water District to vary in different communities from almost nothing to 24.9 gal. per capita of population. James H. Fuertes estimated the volume used for manufacturing to range from 0.4 gal. per capita in the residential town of Wellesley, to 81 gal. in Harrisburg, as given in Table 52, from his report to the Merchants' Association of New York on the waste of water in that city. It must be remembered that these figures are based on the total population of the city and that if all the manufacturing is concentrated in one portion, the per capita consumption figured on the basis of the population of that district would be much higher. The quantity used for manufacturing depends entirely on the number, character, and magnitude of the industries and, whenever possible, an actual canvass and estimate of quantities should be made.

TABLE 52.—SUBDIVISION OF CONSUMPTION INTO VARIOUS USES
Gallons per day per capita
James H. Fuertes, "Report on Waste of Water in New York," 1906

Place	Year	Consumers' use			Public uses	Not accounted for	Total consumed	Per cent unaccounted for	Services metered per cent
		Industrial	Domestic	Total					
Brockton	1904	5.1	15.5	20.6	3.0	13.3	36.9	36	91
Boston	1892	30.0	30.0	60.0	3.0	32.0	95.0	34	
Cleveland	1904	40.0	26.0	66.0	10.0	20.0	96.0	21	49
Fall River	1902	......	......	23.4	8.3	8.7	40.5	21	95
Hartford	1904	3.0	30.0	33.0	5.0	24.0	62.0	39	99
Harrisburg	1904	81.0	30.0	111.0	5.0	30.0	146.0	21	75±
Lawrence	1904	8.0	17.0	25.0	5.0	12.0	42.0	29	87
Milwaukee	1904	45.0	25.0	70.0	5.0	14.0	89.0	16	79
Madison	1904	......	......	21.0	13.0	37.0	71.0	52	96
Syracuse	1904	39.3	31.0	70.3	18.0	20.0	108.3	19	72
Taunton	1904	14.7	21.5	36.2	3.0	24.8	64.0	39	45
Wellesley	1904	0.4	28.6	29.0	2.5	23.5	55.0	43	100
Yonkers	1904	24.0	20.0	51.5[1]	2.0	40.5	94.0	43	100

[1] Total includes 7.5 gal. per capita per day passed through meters at special rate.

The volume used for domestic purposes varies with the class of residence, first-class residences with many fixtures using more per capita than the less elaborate houses, as shown in Table 53.

TABLE 53.—WATER CONSUMPTION PER CAPITA IN HOUSES OF DIFFERENT CLASSES, 1910 OR 1911

Journal of the New England Water Works Association (1913)

City	Apartment houses			First-class dwellings			Middle-class dwellings			Lowest-class dwellings		
	No. of houses	No. of persons	Gal. per day per capita	No. of houses	No. of persons	Gal. per day per capita	No. of houses	No. of persons	Gal. per day per capita	No. of houses	No. of persons	Gal. per day per capita
Baltimore, Md.	...	......	....	...	.....	....	20	126	54	25	84	16
Boston, Mass.	50	2,164	37	40	400	60	50	750	33	50	750	15
Boston, Mass.[1]	...	......	....	...	.....	....	.....	.....	....	50	7,000	24
Cambridge, Mass.	50	1,242	37	50	250	37	50	300	11	50	250	17
Canandaigua, N. Y.	50	248	62	50	290	68	50	180	42	50	146	10
Denison, Tex.	...	......	....	50	153	15	200	799	12	500	3,090	4
Fall River, Mass.	...	......	....	60	328	63	60	457	26	60	1,394	17
Hartford, Conn.	19	560	55	114	659	67	135	1,186	27	98	1,842	24
Hartford, Conn.	75	1,247	24	148	749	43						
Holyoke, Mass.	20	2,215	46	15	92	50	20	113	43			
Holyoke, Mass.	47	2,118	69 = (apartments with stores)									
Pawtucket, R. I.	...	......	....	...	.....	....	482	4,095	26	766	7,188	12
Pawtucket, R. I.	...	......	....	...	.....	....	.....	.....	...	444	4,534	12
Peoria, Ill.	5	150	84	5	30	74	20	80	32	5	15	11
Peoria, Ill.	5	200	63	13	104	47	25	125	28	8	40	6
Plymouth, Mass.	...	......	....	23	94	47	15	67	33	2	4	14
Washington, D. C.	101	3,470	135	84	500	75	100	400	30	100	500	37
Wilmington, Del.	25	500	73	25	189	73	25	125	44			
Worcester, Mass.	50	1,875	60	50	277	42	50	385	66	50	1,179	12
Totals	497	15,989	....	727	4,115	....	1,302	9,188	....	2,258	28,016	
Averages	...	......	62	...	.....	54	.....	.....	34	.....	......	15

[1] Lowest-class dwellings, lower figures are those for tenement blocks containing from 15 to 30 families each.

In some of the largest cities where considerable districts are devoted almost entirely to business and the number of persons in the district during the day, but resident elsewhere, is large, per capita figures of consumption must be studied with care before any conclusions are drawn from them. The figures in Table 54 illustrate this clearly. The subject was investigated by the Metropolitan Sewerage Commission of New York which reported in 1910 that the actual resident population of the Borough of Manhattan was increased about one-third daily by the influx of persons engaged in business pursuits there but residing else-

TABLE 54.—CONSUMPTION OF WATER IN SECTIONS OF MANHATTAN
W. W. Brush, *Proc. Am. Water Works Assoc.*, 1912

Characteristics of district	Consumption, million gal. per day	Resident population	Consumption per capita, gal. per day
Gagings of 1902–1903			
Large hotels, high-class residences	1.87	8,396	223
East Side tenements	1.44	38,906	37
East Side tenements	5.40	90,000	60
Residence and high-class apartments	0.76	10,164	75
Business, office buildings, water-front, shipping	9.45	11,000	860
High-class apartments and hotels	1.37	8,872	154
Uptown residences and medium-class apartments	4.89	4,380	112
Upper East Side tenements, water-front, power houses, and breweries	2.75	39,969	69
Gagings 1911			
East Side tenements, some water-front	11.44	230,500	50
All classes	29.48	204,557	144
High-class apartments and residences	22.18	186,990	118
High-class apartments, residences and tenements	12.74	138,800	92
East Side tenements and water-front	8.28	84,580	98
High-class apartments, residences, tenements and water-front	14.82	173,000	86
All classes	13.38	169,100	79
All classes	13.66	209,393	65

where. The various transportation companies bringing passengers into the borough furnished information to the Commission indicating that 413,500 residents of Long Island, 203,800 of New Jersey, 17,200 of Staten Island and 42,900 from north of the Bronx came to Manhattan daily for business purposes. A somewhat earlier investigation was made by Nicholas S. Hill, Jr., while Chief Eng. of the Department of Water Supply, Gas and Electricity of Manhattan; the results are summarized

in Table 55.[1] The trend toward larger office and mercantile buildings and factories, greater concentration of such buildings, and migration of the population into suburban areas has doubtless greatly accentuated the effects of transient population upon water consumption and corresponding sewage flow.

This influx of nonresidents, which is the cause of greatly increased flow in the sewers serving such districts, doubtless has a corresponding,

TABLE 55.—RESIDENT AND TOTAL POPULATIONS OF CERTAIN DISTRICTS IN MANHATTAN, 1903 (HILL)

District	Resident population	Total population	Increase of total over resident population, per cent	Character of district
1	8,396	12,156	45	Residential and high-class hotel
2	38,906	38,906	0	Tenement houses
3	90,000	90,000	0	East Side tenements
5	32,200	32,450	1	Moderate-priced apartments
6	10,164	10,164	0	Apartment houses; private houses
7	3,076	6,076	98	Gas works; large shops; railroad yards
8	11,000	114,000	937	Office buildings
9	8,872	8,872	0	Apartment houses; private houses

though smaller, effect in the opposite direction upon the flow in sewers serving the districts in which these persons reside. As their residences are widely scattered, however, it is probable that in no place will the reduction in flow be sufficient to warrant any allowance for it in design, although it is very important to provide for the increased flow in the sewers serving the business districts into which they go.

Ratio of Sewage Flow to Water Consumption.—The North Metropolitan sewerage system of Boston furnishes valuable information regarding the relations between the quantity of sewage reaching a large intercepter, and the population, area, water consumption, and rainfall of the district served. The average relations by years between the quantity of sewage and the water consumption are given in Table 56. In this table, the last column shows the relation of the volume of sewage to the estimated water consumption in the area served by the sewers.

[1] *Eng. News*, 1903; **49**, 335.

The relations of the maximum to the average monthly sewage flow and to the average monthly water consumption are shown in Fig. 63. The circles representing rates of flow on days of maximum flow must not be misinterpreted, for the sewer is protected by storm outlets, which permit the discharge of much of the flow, unmeasured, at such times.[1] The relations between the sewage flow and the water consumption for the dry period of each year from 1904 to 1912 are given in Table 57. The dry months were selected in the driest season of the year and after a

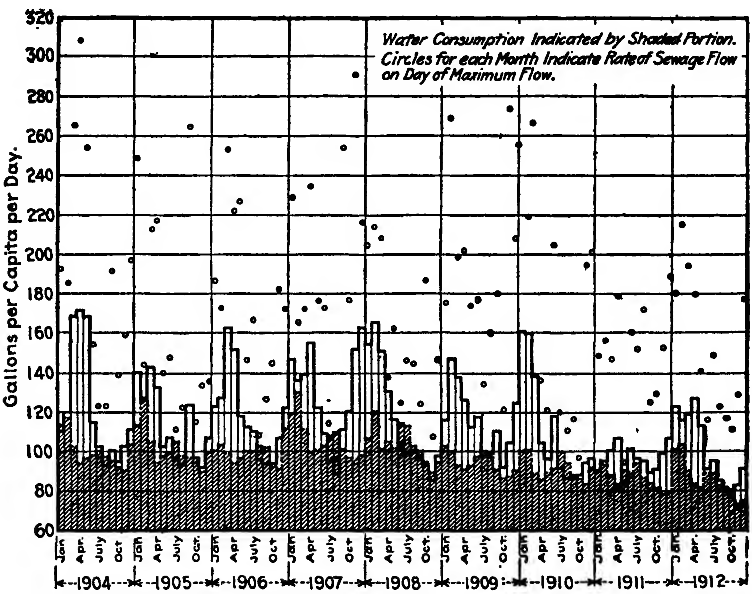


FIG. 63.—Flow of sewage and water consumption in North Metropolitan Sewer District, Boston, 1904–1912.

month of dry weather; in a few cases they show a rather high rainfall, but it was concentrated in a few days so as to leave the month a dry one as a whole. It will be noticed that there are no figures for 1911; none of the monthly records in that year is representative of dry weather conditions. In fact the 1908 and 1909 figures are probably high on account of some rain-water in the sewage, the rainfall figures in the table making this appear probable.

[1] These average annual figures are open to the criticism that they include some storm water, for some of the local sewers discharging into the intercepters are on the combined system. Nevertheless, the figures are fairly representative of the sewage flow as influenced by infiltration. As shown later, it is estimated that during dry weather, if no storm water enters the sewers, the sewage flow will be about 90 per cent of the water consumption.

Table 56.—Relation of Annual Average Quantity of Sewage to Water Consumption. North Metropolitan Sewer District, Boston, Mass.

Year	Precipitation in inches at Chestnut Hill	Based on total population of district			Based on population *connected* with sewer system	
		Average sewage flow, gallons per capita per day	Average water consumption, gallons per capita per day	Ratio of sewage flow to water consumption, per cent	Average sewage flow, gallons per capita per day	Ratio of sewage flow to water consumption, per cent
1904	43.40	122	100	121	156	155
1905	40.84	114	102	111	139	137
1906	47.16	118	100	119	150	150
1907	51.83	128	106	121	152	143
1908	43.31	117	105	111	138	131
1909	47.62	116	95	122	134	141
1910	39.05	110	92	119	127	137
1911	41.28	97	87	112	110	127
1912	39.96	100	87	115	113	130
1913	43.29	99	79	126	111	141
1914	38 99	101	79	127	112	142
1915	44.92	100	72	140	111	154
1916	41.91	109	75	145	121	161
1917	43.62	105	77	137	117	152
1918	39.81	107	87	123	120	138
1919	48.15	112	79	143	125	158
1920	50.56	117	82	143	129	157
1921	45.44	107	77	139	117	152
1922	49.10	111	79	141	121	153
1923	41.46	114	87	136	124	148
1924	38.81	110	82	134	119	145
1925	45.00	112	83	135	121	146
1926	39.82	112	83	135	121	146
Average..	43.71	110	86	129	124	145

The average ratio of dry-weather sewage to water consumption is given in Table 57 as 94 per cent. This would probably be reduced by 3 or 4 per cent if it were possible to exclude storm water entirely from the sewage flow. This adjustment was estimated from a graphical study, by the authors, of detailed record sheets of the Metropolitan sewerage works

for seven typical months during 1904 to 1909 inclusive. From a study of the diagrams of sewage flow during each month, it seems probable that the true ratio of sewage flow to water consumption is about 90 per cent, if the entire amount of storm water is excluded. If this estimate is in error, it is probably too high, as the quantity of ground water assumed in rounding off the figures is only about 1,200 gal. per mile of sewer, this being in extremely dry weather. This ratio, it must be kept in mind, is a purely local one and can hardly be expected to agree with other conditions than those on which it is based. Unfortunately, very little information of this nature is available, and engineers having opportunities to keep such records should not neglect to do so.

TABLE 57.—RATIO OF SEWAGE FLOW TO WATER CONSUMPTION DURING DRY WEATHER. NORTH METROPOLITAN SEWER DISTRICT, BOSTON, MASS.

Year	Month	Rainfall (inches) at Chestnut Hill		Average sewage flow, gallons per capita per day	Average water consumption, gallons per capita per day[1]	Ratio of sewage flow to water consumption, per cent
		For month	For month previous			
1904	August	2.74	1.48	94	98	96
1905	August	3.47	1.92	94	98	96
1906	September	2.92	1.82	95	102	94
1907	August	1.79	1.49	89	110	81
1908	October	4.34	1.22	93	96	97
1909	August	4.11	1.10	99	98	101
1910	August	1.18	1.93	87	94	92
1910	September	2.65	1.18	86	89	97
1912	September	1.72	2.24	80	83	96
1912	October	1.61	1.72	77	81	95
Average		2.65	1.61	89	95	94

[1] Based on total population of district.

The ratio of 90 per cent does not mean literally that 90 per cent of the water supply is delivered to the sewer, but rather that the dry-weather flow of sewage bears that relation to the supply.

Ratio of Sewage Flow to Area.—For use in the design of sewers, it is generally more convenient to utilize estimates of sewage flows in gallons per day per acre than in any other form. The direct estimation of flow in this form is not advisable, however, because the effects of character of development, of population density and the like are not so apparent

and there is greater danger of inappropriate figures being used without detection of their unsuitability.

By estimating the possible density of population likely to be experienced in various areas and the sewage flow per capita, figures of sewage flows per acre for areas of various sizes are readily computed, in form for use in sewer design. The method of doing this is illustrated in Chap. VI.

Table 63 on page 215 contains figures of the actual measured flow of sewage in dry weather for various districts of Chicago, both in totals and in quantities per acre and per square mile.

ADDITIONS TO THE SEWAGE

Private Water Supplies.—Some sewage reaches the sewers from those hotels, public baths, and other buildings where the public water supply is supplemented with water from wells. In addition, where industrial wastes originating from the use of water derived from private sources are common, the volume of water obtained from private supplies and discharged after use into the sewers can usually be learned from the parties using such supplies, or from inspection of their plants, although it is sometimes necessary to gage the water used or the sewage discharged in order to obtain trustworthy figures.

Ground Water.—The term "ground water" is employed in sewerage practice to mean not only all water in the pores of the materials through which sewers are laid, but also the surface water leaking into sewers through perforated manhole covers and defective manhole masonry. Where the sewers are on the combined plan, ground water also includes the dry-weather flow of any small brooks connected with the system. From half to three-fourths of the rainfall usually runs off very quickly into the storm-water drains or combined sewers, or directly into streams, and the remainder percolates into the ground, becoming ground water. The presence of ground water in the earth about the sewers results in leakage into them.

The sewers first built in a district usually follow, in a general way, the natural water courses, and therefore lie in the bottoms of the valleys. Such sewers, especially in case of combined systems, often are built very close to, or actually in, the natural beds of brooks. They are not usually extended to the extreme upper end of the district at first, and consequently the natural runoff through these brooks is taken into the sewers. Such brooks frequently flow with gradually diminishing volume for many days after the immediate runoff from a storm has passed by, and perhaps even throughout the dry season. The flow during the remainder of the time until the next storm is made up of the water draining out of the land and is therefore logically classed with ground water

and, as its flow is continuous though gradually diminishing, it has the same effect upon the quantity of sewage. As a result of these conditions, such sewers receive comparatively large quantities of ground water, while it is but natural to expect that sewers built in these districts in later years, necessarily at higher elevations, will receive smaller quantities of leakage and brook flow. Moreover, as the paved and built-over area increases, the water falling upon the surface runs off more rapidly through the water courses, drains, or combined sewers, and leaves less to percolate gradually through the ground and thus to find its way into the sewers by infiltration or leakage.

Many measurements have been made to determine the quantity of ground water which finds its way into sewers. The results of these observations indicate that the maximum quantity of infiltration may be as low, under the most favorable conditions, as 5,000 or 10,000 gal. per day per mile of sewer. On the other hand, they show that the leakage sometimes amounts to from 20,000 to 40,000 gal. per day per mile of sewer and at times of very high ground water, or during rain when there is leakage through manhole covers, even in separate systems, it may run as high as 100,000 gal. per day per mile of sewer. In fact, there are instances where leakage has materially exceeded this quantity.

As a rule, there has been a growing tendency toward securing as nearly watertight construction as possible, and it may be true that the older systems receive greater quantities of ground water than some of the better-constructed modern systems.

Leakage.—The amount of ground water which finds its way into the sewers is called "leakage." It is a very variable part of the flow in the sewers, depending on the quality of the materials and workmanship employed in the original construction, on the degree of care in maintenance and in preventing damage to the sewers by drain layers or plumbers when making building connections, and on the height of the ground-water table.

In the case of the North Metropolitan (Boston) intercepter, already mentioned several times in this chapter, it is possible to form a fairly close estimate of the amount of this leakage, for if 90 per cent of the average monthly water consumption is equivalent to the sewage flow at the same time, by subtracting this quantity from the measured sewage flow, the remainder will be the infiltration into the sewers. As this leakage will be greatest in very wet weather, the figures for the wettest period of each year have to be studied, and the results of such a study in this case are given in Table 58.

The amount of leakage is stated in different ways by different engineers, as so much per unit length of pipe, per capita, or per acre. It depends, of course, on the length of pipe, and to a certain extent on the population, which affects the number of connections and the lengths

of the sewers and the consequent opportunity for leaks. In Table 75 are shown the allowances made for leakage in the designs for various cities and in Tables 59, 60, and 61 the actual measurements of leakage at certain places.

A paper on the "Infiltration of Ground Water into Sewers" by John W. Brooks[1] enumerates the factors influencing infiltration, as follows: (1) the diameter and length of the sewer; (2) the material of which the sewer is constructed, and (a) in vitrified pipe sewers, the type of joint used, (b) in concrete or brick sewers the type and quantity of waterproofing used; (3) the skill and care used in laying the sewer; (4) the character of the materials traversed by the sewer; (5) the relative positions of the sewer and the ground-water level. After discussing the various units such as gallons per day per capita or per mile of pipe, he suggests the following units: for vitrified pipe, gallons per day per foot of joint; for concrete and brick sewers, gallons per day per square yard of interior surface.

TABLE 58.—LEAKAGE IN NORTH METROPOLITAN SEWER DISTRICT, BOSTON, IN APRIL AND MAY

	Average	Maximum	Minimum
Gallons per capita per day	62.2	93.8	38.7
Gallons per acre per day	1,738	2,577	1,094
Gallons per mile of sewer per day	50,600	78,900	30,900

In the discussion of the paper, John H. Gregory suggested as a unit the number of gallons per day per inch of diameter per mile of sewer. S. L. Christian stated that observations practically checked previous assumptions as to the quantity of ground water to be provided for at New Orleans, where all of the sewers are below the ground-water level. He stated that the leakage in gallons per day per mile of sewers was as follows: 1907, 55,000; 1908, 53,000; 1909, 51,000; 1910, 51,000; 1911, 48,000; 1912, 42,000. E. G. Bradbury questioned the value of a unit based on the diameter of the sewer, as but very few sewers are sufficiently watertight to prevent the lowering of the ground water in the vicinity to the level of the pipe. He was of the opinion that most sewers permit the entrance of ground water about as fast as it gets to them.

Thomas McKenzie[2] reports that at Westerly, R. I., the average leakage of ground water was 30,000 gal. per day per mile of sewer, or 600 gals. per acre per day.

[1] *Trans. Am. Soc. C. E.*, 1913; **76**, 1909.
[2] *Jour. Boston Soc. C. E.*, 1925; **12**, 140.

TABLE 59.—LEAKAGE OF GROUND WATER INTO SEWERS

Place	Gal. per day per mile of sewers	Extent of sewers considered
Alliance, Ohio	195,000	
Altoona, Pa	41,000	1.2 miles
Altoona, Pa	86,000	0.6 mile
Altoona, Pa	264,000	0.95 mile
Brockton, Mass	45,000	2,000 ft.
Brockton, Mass	61,000[1]	10,400 ft.
Brockton, Mass	178,000[2]	10,400 ft.
Canton, Ohio	26,000	11 miles
Clinton, Mass	32,500	
Concord, Mass	30,000	whole system
East Orange, N. J	22,000[3]	29 miles
East Orange, N. J	9,000	25 miles
Framingham, Mass	35,000	whole system
Gardner, Mass	45,000	whole system
Joint Trunk Sewer	25,000[4]	150 miles
Madison, Wis	48,000	
Malden, Mass	50,000	whole system[7]
Marlboro, Mass	50,000	whole system
Medfield, Mass	25,000[5]	whole system
Metropolitan System, Mass	40,000[6]	137 miles
Natick, Mass	80,000 to 100,000	8.58 miles
New Orleans, La	32,000 to 60,000	
North Brookfield, Mass	24,000	1.41 miles
Peoria, Ill	100,000	
Reading, Pa	5,000	
Westboro, Mass	1,072,000	3,010 ft.
Worcester, Mass	32,000	

[1] Water in river low. [2] Water in river high. [3] Great precautions taken to prevent leakage, as construction was carried on in quicksand and the ground-water table was naturally 10 ft. or more above the sewer. [4] This relates to the sewer serving parts of Newark and Elizabeth, N. J., and smaller places westward to Summit. [5] Before house connections were made. [6] Before any connections were made. [7] 38 out of 45 miles total.

The Malden figures are from *Eng. News*,[1] the Concord figures from the 1900 report of the Sewer Commissioners and the remainder from reports of the Mass. Board of Health and *Trans. Am. Soc. C. E.*[2]

[1] *Eng. News*, 1903; **50**, 180.
[2] *Trans. Am. Soc. C. E.*, 1913; **76**, 1909 *et seq.*

TABLE 60.—INFILTRATION INTO SEWERS AT VARIOUS PLACES

Place	Sizes	Miles of sewer	Miles below ground water	Infiltration: Gallons per day per mile	Infiltration: Gallons per day per mile below ground water	Kind of joints	Remarks
Long Beach, Calif.	8.10	5.67	2.73	12,600	26,200	Cement	Tested before house connections were made
Long Beach, Calif.	8.18	12.41	3.38	1,780	7,450	Cement	
Long Beach, Calif.	8.24	13.60	1.71	690	5,490	Cement	
San Luis Obispo, Calif.	....	.....	....	30,000			
Santa Cruz, Calif.	....	.....	....	......	2,110[1]	....	4 gal. per day per inch diameter per 100 ft.
Monte Vista, Col.	....	8	....	650,000	....	Cement	80 per cent of total flow
New Haven, Conn.	....	.....	....	18,000			
St. Petersburg, Fla.	....	.....	....	13,000			
Kewanee, Ill.	....	.....	....	10,000–15,000			
Detroit, Mich.	....	.....	....	......	....	....	1,000 gal. per acre daily
Billings, Mont.	....	.....	....	......	....	....	1,500 gal. per acre daily
East Orange, N. J.	....	.....	....	27,000			
Rutherford, N. J.	....	.....	....	10,000			
East Aurora, N. Y.	....	.....	....	2,000			
Goldsboro, N. C.	....	.....	....	......	....	....	⅓ capacity of sewer
Akron, Ohio	....	.....	....	1,000			
Salem, Ohio	....	.....	....	12,000			
Altoona, Pa.	8.12	2.8	....	2,140	....	Compounds	
Charleston, S. C.	....	.....	....	46,000 to 18,000			
Jacksonville, Tex.	....	.....	....	......	....	....	50 per cent of flow
Salt Lake City, Utah	....	.....	....	......	40,000±	....	0.061 cu. ft. per second per mile, before connections
Seattle, Wash.	....	.....	....	......	....	....	1 gal. per minute per acre
Kitchener, Ont.	....	.....	....	10,000			

This table has been prepared from data compiled from the answers to a questionnaire in "Public Works," 1927; 58, 363.

[1] Assuming that average diameter is 10 in.

F. W. Haley[1] states that at Framingham, Mass., the leakage into 3.06 miles of pipe sewers before connections were made amounted to 500 gal. per day per mile; or, assuming all the leakage to occur in 2,000 ft. of 10- and 18-in. sewer which was below the elevation of the river, 4,000 gal. per day per mile. In another section of 4.75 miles, none of which was below ground water, there was no leakage.

Carl H. Nordell,[2] describing weir gagings of sewage flow at Milwaukee, stated that the average flow in residential districts was equivalent to 64 gal. per day per person; and as the meter records showed a water consumption of 31 gal. per day per person, which he assumed to equal the domestic sewage, the remaining 33 gal. per day per person was considered as ground-water leakage.

In general, the authors have found that water finds its way into sewers through defective joints in pipes or brick structures, through concrete which is porous, and through cracks due to contraction or other causes.

TABLE 61.—ACTUAL FLOW OF DOMESTIC AND INDUSTRIAL SEWAGE AND GROUND WATER AT TOLEDO, OHIO, 1917

Sewer district	27	26	16	22	Totals and means
Population, total	2,634	3,902	17,087	17,838	41,461
Connected to sewers	1,889	3,673	16,842	16,308	38,712
Area, acres	280	288	1,083	502	2,153
Length of sewers, miles	6.73	5.96	17.82	11.70	42.21
Domestic sewage:[3]					
Gallons per day per capita					
of total population	20	19	34	10	21
of population connected to sewers	28	20	34	11	23
Industrial sewage:[3]					
Gallons per day per acre	4,900	4,000	20,000	127,000	14,000
Ground water:[4]					
Gallons per day per acre	557	736	1,100	246	783
Gallons per day per mile of sewer	23,400	35,600	67,000	10,000	40,000

Data are from weir measurements of flow of sewers discharging into Ten Mile Creek as reported by Watson G. Harmon.[5] Measurements made during dry weather.

These imperfections are sufficiently numerous and large to allow the infiltration of water to such an extent that the water table at the sewer

[1] *Jour. Boston Soc. C. E.*, 1925; **12**, 258.
[2] *Eng. News-Record*, 1917; **65**, 78.
[3] Water supply metered to users.
[4] Minimum night flow less simultaneous industrial sewage.
[5] *Eng. News-Rec.* 1918; **80**, 1233.

rarely lies above its crown and usually is found near the invert, although its elevation varies greatly with the quantity of rain and snow water percolating into the ground. This is usually greatest in the northern part of the country in the spring of the year, when the frost coming out of the ground leaves it porous so that the water from slowly melting snow and ice and from gentle long-continued rains may readily percolate through the upper strata which later in the year form a hard compact crust more nearly impervious.

It is often held that sewers which at first are porous or have small cracks and poorly filled joints will gradually "silt up;" that is, the pores will become filled with particles of fine clay and sand and the leakage will thus be reduced. Trenches also become compacted, and if in clay, a nearly watertight layer may be formed around the sewers, thus cutting off the water so that it will not follow along the pipes and enter through imperfect joints. These observations are all more or less well founded, but it is also a fact which largely and sometimes more than offsets the foregoing causes of reduced leakage, that many times the pipes crack after being laid and that connections made from time to time are so poorly constructed that they are the source of considerable leakage. Abandoned connections are rarely sealed at the sewers and may admit much water. Manholes are "heaved" by the frost so that water may enter between the courses. The net result of these changing conditions appears to be the presence of a gradually increasing quantity of ground water in the sewers.

As the water does not usually percolate or leak into sewers entirely around their perimeters, but rather enters near the water line, it seems hardly logical to report leakage in terms of area of masonry surface, of length of pipe joints, or even of radius or diameter. It is doubtful even if the number of pipe joints per mile throws much light on the subject, although the chances of poor joints in the main sewer are proportional to the number of joints. This, however, takes no account of the leakage through house connections.

Data are most easily obtained in terms of quantity of leakage per mile of sewer, and the most leakage may enter the smallest sewers. Having the data in this unit, it may for convenience be calculated in quantity per capita and quantity per acre, the latter being probably the most convenient form for use in planning intercepters and trunk sewers and in studies for pumping stations and treatment works. For detailed computations of small lateral sewers, the quantity per capita is perhaps most readily used.

The authors believe that an effort should be made to secure data in at least these three terms, gallons per mile of sewers, gallons per capita of population residing within the district served, and gallons per acre of this district.

ACTUAL MEASURED FLOW OF SEWAGE

Statistics of Sewage Measurements.—Table 61 above contains figures of the flow of sewage in a portion of the city of Toledo, Ohio. The gagings were made to obtain data upon which to base the design of an intercepting sewer. The bases finally assumed were:

Ground water, 1,200 gal. per day per acre.

Industrial sewage, 14,000 gal. per day per acre.

Domestic sewage $\left(1 + \frac{14}{4 + \sqrt{P}}\right)$ 40 gal. per day per capita, where P = population in thousands.

In Table 62 are given statistics of the sewage flows of a number of Massachusetts cities and towns. These communities all have sewerage systems on the separate plan and the flows consequently are unaffected by storm water except as it increases the leakage and the amounts of roof and surface water improperly discharged into the sewers. It should be noted further that they are mostly flows from small communities without large quantities of trade wastes, and that the volumes per capita are much smaller than those to be expected in large cities.

TABLE 62.—MAXIMUM AND AVERAGE FLOWS OF SEWAGE, 1903
Massachusetts State Board of Health

Place	Population	Average yearly quantity of sewage				Average quantity of sewage in max. month			
		Gallons per 24 hours				Gallons per 24 hours			
		Per inhabitant	Per person connected	Per connection	Per mile of sewer	Per inhabitant	Per person connected	Per connection	Per mile of sewer
Andover	7,214	17	35	290	11,600				
Brockton	44,202	20	35	512	26,930	31	55	799	41,990
Clinton	14,969	52	78	528	40,900	78	117	787	60,940
Concord	5,938	53	260	1,311	41,430	77	379	1,912	60,420
Framingham	12,376	53	87	537	41,400	78	129	796	61,400
Gardner	......	.....	86	1,090	37,750	.....	161	2,032	70,375
Gardner	11,792	47							
Templeton	......	.....	56	714	33,780				
Hopedale	2,513	60	75	750	37,500				
Leicester	3,522	9	60	429	14,020				
Marlborough	12,788	86	110	691	45,450	159	203	1,274	83,800
Natick	9,892	57	142	893	52,400	113	280	1,765	103,510
Pittsfield	22,519	65	97	797	45,930	69	104	854	49,240
Southbridge	11,090	32	159	1,108	61,400				
Spencer	7,635	49	125	625	37,500				
Stockbridge	2,083	36	94	700	21,430				
Westborough	5,499	51	94	1,007	38,900	104	190	2,039	78,760

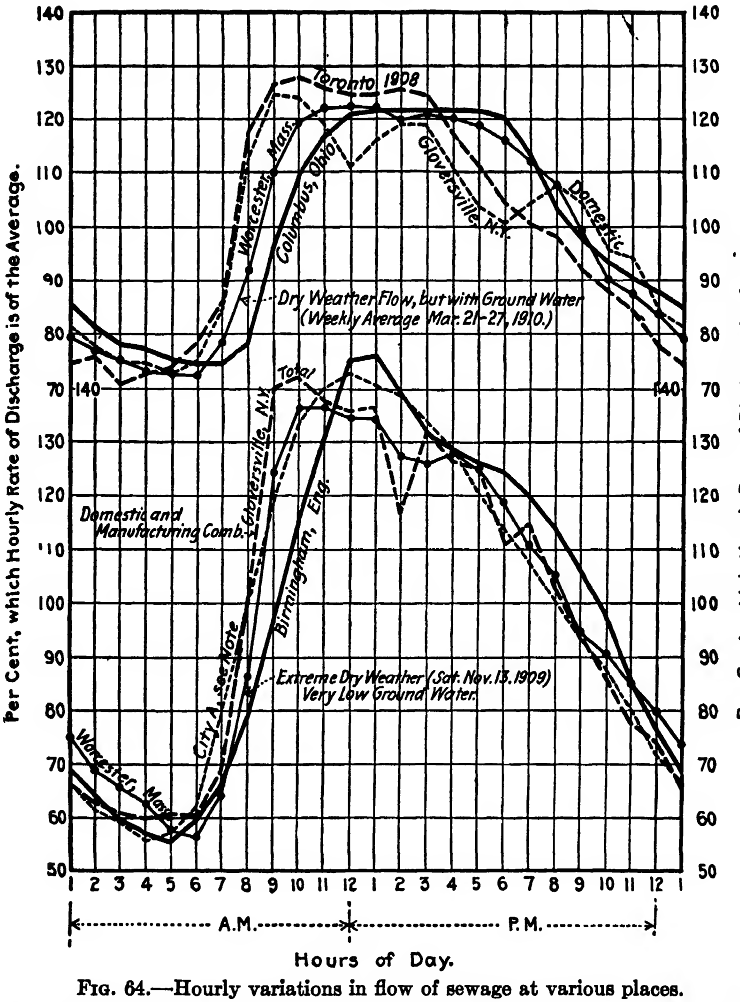


FIG. 64.—Hourly variations in flow of sewage at various places.

Worcester, Mass.—Estimated population, 150,000; average dry-weather flow, 15,300,000 gal. per day; 1½ hours period of flow from city.

Toronto, Ont.—Dry-weather flow, 73 gal. per capita per day; gagings made in city, October, November and December; from Report of City Eng., 1908.

Columbus, Ohio.—Discharge of intercepting sewer from weir measurements at the outfall, Dec. 2 to 9, 1904; average week-day flow during extremely dry weather = 8,400,000 gal.; estimated population in 1905, 150,000; from Johnson's Report on Purification of Columbus Sewage.

City A (name must be omitted for local reasons).—From city to outlet is about 3 hours period of flow; population, 15,000; typical average dry-weather flow in July and August, 1908, 300,000 gal per day.

Birmingham, Eng.—Gagings at treatment works; manufacturing wastes about one-fifth of dry-weather flow: two years, 1906-7.

Gloversville, N. Y.—Mill wastes about 26.2 per cent of total flow; population, 20,000; average flow 2,000,000 gal. daily; gagings at experiment station, with half-hour period of flow from city; Oct. 30, 1906.

Quite a different result is to be noted in certain sewer districts of Chicago, as indicated in Table 63. The sewers are on the combined plan, but the measurements were made during dry weather when the sewage presumably contained no storm water. The excessive flow in these sewers is to be accounted for largely by the great consumption of water in the city, which in 1910 averaged 242 gal. per day per capita of the population.

Fluctuations in Flow.—The flow of sewage fluctuates between wide limits and follows somewhat the variations of the consumption of water. The day flow is also increased by the greater discharge of industrial wastes at that time. During the spring or wet months, the flow is increased by the added volume of ground water contributed, some of

TABLE 63.—TYPICAL SEWERED AREAS AND DRY-WEATHER RUNOFFS, CHICAGO, 1910 AND 1911

Data from Wisner's Report on Sewage Disposal, Sanitary District of Chicago, 1911

Sewer outfalls	Drainage area in acres	Population estimated 1911	Dry weather runoffs				Density of population per acre	Period covered by observation
			Million gallons per day	Gallons per day per acre	Million gallons per day per square mile	Gallons per capita per 24 hours		
Diversey Boulevard (W).	890	23,550	5.59	6,300	4.02	238	26.4	Aug. 15 to 17, 1911 2 days
Randolph St. (W)..	240	11,368	3.95	16,400	10.5	348	47.4	Aug., 3 days
Robey St. (S)......	2,500	38,728	6.5	2,600	1.67	169	15.5	June 1 to 3, 1911, 2 days
Ashland Ave., (S)..	980	44,581	15.0[1]	15,300	9.8	338	45.5	May 18 to 20, 1911, 2 days
Center Ave., (S)...	660	23,463	13.5[2]	20,500	13.1	578	35.6	May 16 to 18, 1911, 2 days
Thirty-ninth St. pumping station.	14,340	285,900	90.6[3] 64.6[4]	6,300 4,500	4.04 2.88	318 227	20.0	
Ninety-second St.	98	3,666	1.19	12,200	7.8	325	37.4	209 days Aug. 1, 1910 Mar. 31, 1910
Wentworth Ave., (S), (Calumet).	5,300	30,464	8.0	1,500	0.97[5]	264	5.8	Aug. 1, 1910 July 31, 1911 253 days

[1] Daily variation average:
8 A. M. to 8 P. M. 18.4 million gallons daily, contains large amount of industrial waste.
8 P. M. to 8 A. M. 12.0 million gallons daily.

[2] Daily variation average:
8 A. M. to 8 P. M. 16.6 million gallons daily, contains large amount of industrial waste.
8 P. M. to 8 A. M. 11.0 million gallons daily.

[3] This runoff or more for 76 days in 1909.

[4] This runoff or more for 276 days in 1909.

[5] 1.5 million gallons daily per square mile occurred 329 days in the year.

which is present at all times in most sewerage systems. In Fig. 64 are plotted the flows of sewage in terms of percentage of the average, from a number of cities. An attempt has been made to synchronize the curves by making allowance for the time required for the sewage to flow from

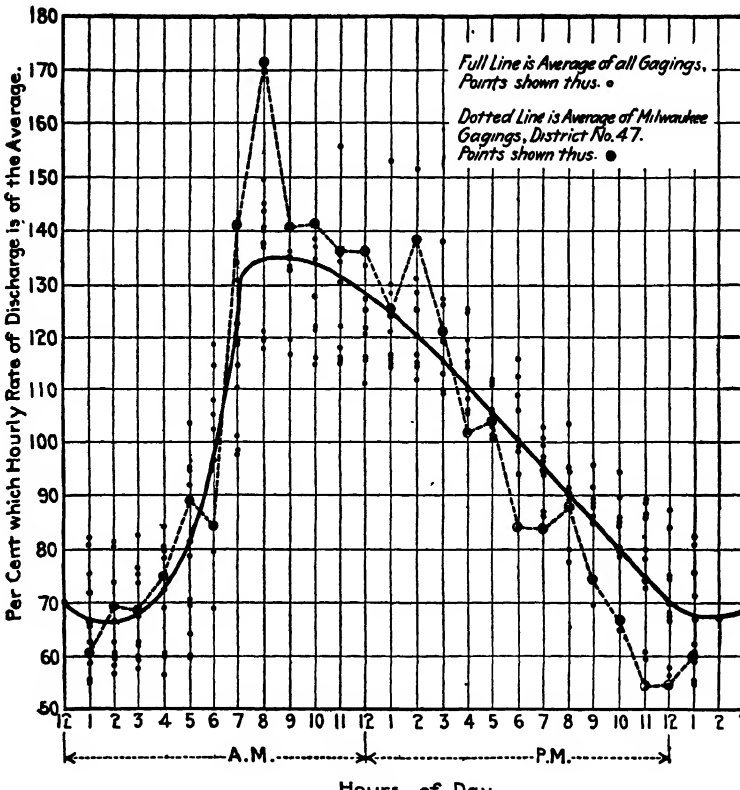


FIG. 65.—Hourly variation in flow of sewage in various cities.

The records used in preparing this diagram were from the following sources: Birmingham, England, average of two years, 1906–7; East Orange, N. J., March 16–17, 1910; Gloversville, N. Y., Oct. 30, 1906, and Sept. 12, 1907; City A, of 15,000 population, typical average curve; Milwaukee, Wis., Oct. 24–28, 1910; Toronto, Ont., 1900 and 1908; Worcester, Mass., Nov. 13, 1909, and March 21–27, 1910.

the city to the gaging point. The curves on the lower part of the figure are typical of dry-weather conditions when ground water is at a minimum, while the curves on the upper portion of the figure are typical of conditions when ground water is relatively high.

Two curves are shown in Fig. 65, taken from the report of the Sewage Disposal Commission of Milwaukee, 1910. The dotted line represents

the flow from a large residential sewerage district in Milwaukee. The smooth curve is drawn through points obtained by averaging points taken from several curves representing the flow from the cities named in the note accompanying the illustration. In this case the curves were synchronized and an effort made to produce a curve typical of the fluctuations in flow of the sewage from the larger cities.

S. M. Cotten[1] found that the curve of distribution of sewage flow at Phoenix, Ariz., was very similar to the average curve on Fig. 65. He also found that the maximum rate of flow from a purely residential district, including the greater part of the city, occurred on Sunday instead of Monday as is usual in northern cities. The amount of sewage

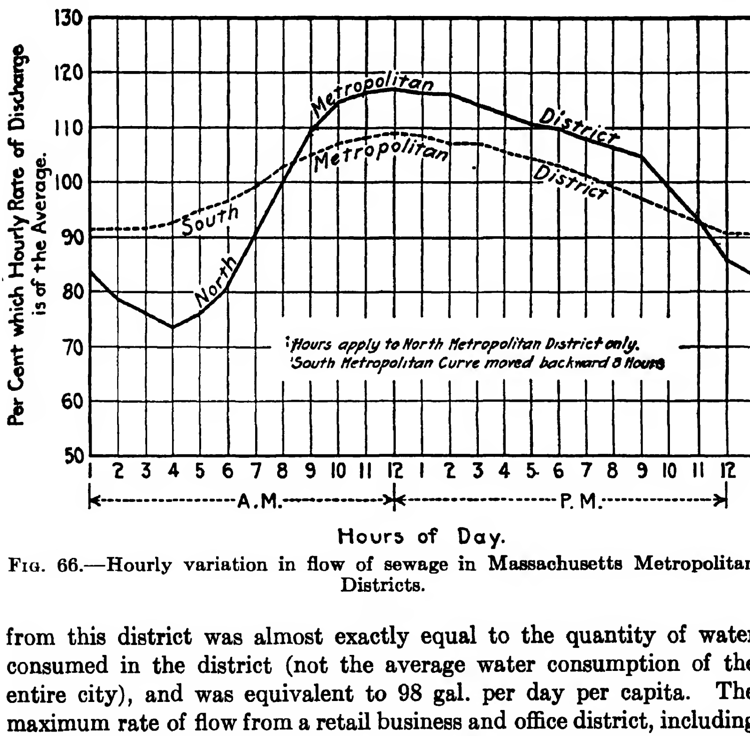


FIG. 66.—Hourly variation in flow of sewage in Massachusetts Metropolitan Districts.

from this district was almost exactly equal to the quantity of water consumed in the district (not the average water consumption of the entire city), and was equivalent to 98 gal. per day per capita. The maximum rate of flow from a retail business and office district, including hotels, was equivalent to 171 gal. per day per capita of the population residing or doing business therein.

Sewage flow gagings at Austin, Tex., made in 1916 by Julian Montgomery,[2] also showed a daily fluctuation in flow similar in general to that shown by the average curve on Fig. 65, but the fluctuation was wider

[1] *Eng. News-Rec.* 1922; **89**, 837.
[2] *Eng. News*, 1917; **77**, 57.

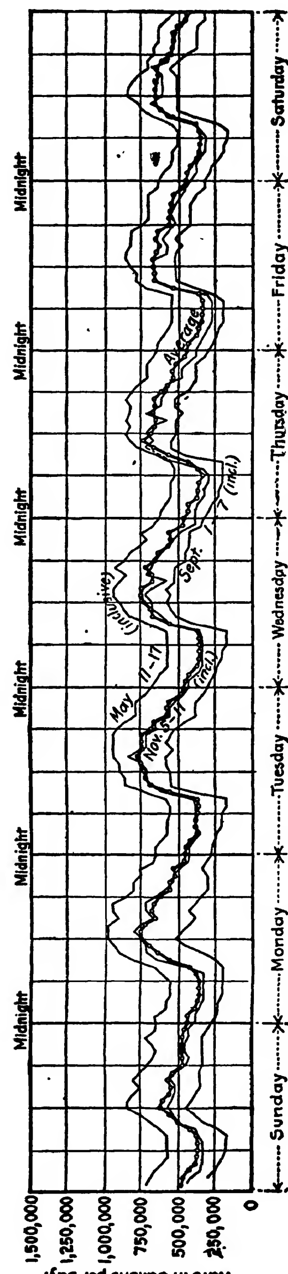


FIG. 67.—Fluctuations in quantity of sewage reaching treatment works of a city of 15,000 population. The fluctuations indicated by the curves occur in the city from 2 to 3 hours earlier than at the works.

(from a minimum of 55 to a maximum of 155 per cent) and there was a subpeak in the afternoon following a depression at noon.

Obviously the fluctuations will be greater in single lines of sewers, or in small districts, than in trunk and intercepting sewers serving large areas. The fact that the sewage requires a longer time to flow from

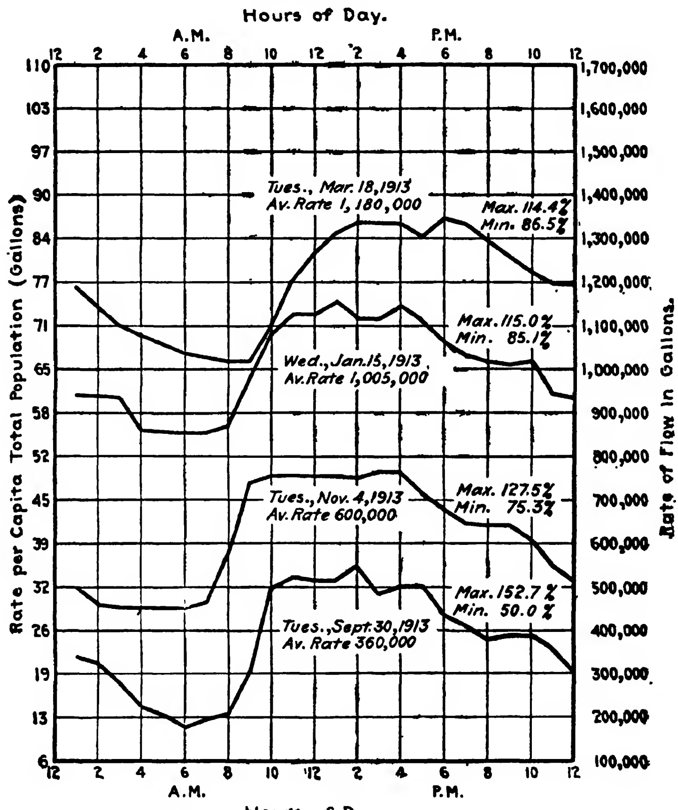


FIG. 68.—Hourly fluctuations in rate of flow of sewage containing different proportions of ground water.

certain districts than from others assists in producing a more nearly uniform flow in the intercepting sewers, as is evident from Fig. 66.

The flow on different days of the week varies considerably. In general, on Sunday the quantity is smaller, and on Monday larger, than on other days. On Monday the rate of the maximum flow is usually somewhat higher than on other days. The typical curve of sewage flow for one week in a city of about 15,000 population is given in Fig. 67.

This city will be termed City *A* in this discussion, as the authors are not permitted to give its name.

The rate of infiltration of ground water varies greatly from season to season, but does not usually fluctuate materially from hour to hour. As the proportion of ground water increases, the fluctuations in the total quantity of sewage flowing from hour to hour naturally decrease. This is illustrated by Fig. 68, showing typical curves of hourly flow of sewage at City *A*, with flows ranging from 360,000 to 1,180,000 gal. per day, the excess of the larger flows being due wholly to ground water. From similar data the curve given in Fig. 69 has been prepared, illustrating

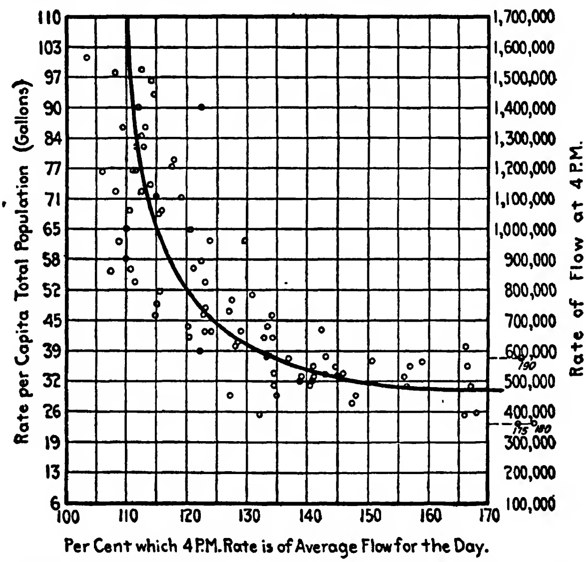


FIG. 69.—Ratio of flow of sewage at 4 P. M. to average flow per day in a city of 15,000 population.

a method by which it is possible to calculate the average rate of flow on any day when the flow at a given hour in the day is known. While this curve is not applicable to other cities, it illustrates a convenient method of obtaining fairly reliable records of the average quantity of sewage, by single daily observations. It is not as satisfactory as the use of a recording gage and should not be employed where the latter is available.

Variations in Quantity of Sewage from Districts of Different Types.—The quantity of sewage to be expected from a district depends upon its character. A residential district will produce sewage made up of household wastes and ground-water leakage, the former being governed by the quantity of water consumed, which will vary from 10 or less gallons

per day per capita in the lowest class dwellings to 75 gal. in first class dwellings or to 135 gal. in apartment houses, as shown by Table 53, page 200. A mercantile or commercial district will yield a much greater quantity on account of the great office buildings where water is used for many purposes, such as the operation of lavatories, motors, and elevators. The flow from such districts will consist of the used water from the municipal supply, the ground-water infiltration, and in many places the used water pumped from wells, which often amounts to a large quantity. Manufacturing or industrial districts may contribute large quantities of liquid wastes. Some of this water is derived from the municipal supply, but frequently very large quantities are taken from wells, rivers, lakes, or even from salt water. The sewage from such districts is, therefore, made up of the used municipal supply from residences and industrial establishments, of the used private supplies of the manufactories, and of ground water. On the other hand, areas made up of parks and cemeteries often contribute only ground water.

Classification of Areas.—A rational classification of areas in a city is a matter for careful study, due consideration being given to such natural conditions as topography and proximity to rivers, lakes, or tidewater, and to such artificial conditions as railroad and street-car lines, docks, and canals. The residential districts usually occupy the uplands and sections topographically unsuited to industrial works. The commercial or mercantile districts occupy the more level areas in the "center" of the community, usually convenient to railroad terminals and docks, and contain public and office buildings, retail and wholesale stores, depots and freight houses, hotels, theaters, and generally some apartment houses. The commercial area is usually relatively small, and while provision should be made for future growth, care must be exercised to prevent estimating too large an area, for the unit quantities of sewage are large. Industrial areas are generally located on fairly level ground along the railroad lines, where spur tracks and sidings may be had. Works using large quantities of water are likely to be located along rivers or near docks where cheap water supplies may be had. These areas also are relatively small.

Philadelphia Sewer Gagings.—Gagings of the dry-weather sewage flow from Philadelphia districts of different types of development were described in the annual report for 1912 by W. L. Stevenson. Some of the data procured are given in Table 64.

Residential Districts.—In computing the probable quantity of sewage from residential districts, it is first necessary to estimate the population likely to reside in them and decide upon the number of persons per acre for which provision should be made. In doing this, it is not safe to assume the same density as that estimated for the entire city, which rarely runs over 25 persons per acre, according to Table 45. The

density in a particular district may run much higher; for example, one ward in Boston had a density of 190 in 1910 (Table 46, p. 188), and there are in most large cities small sewer districts in which the density of population greatly exceeds 25 per acre. The flow from the district is readily calculated from the area, density of population, allowances for maximum rate of used water supply, and maximum rate of ground water.

In Cincinnati in 1912 the flow in a number of sewers serving residential, mercantile and industrial districts was gaged under the general

TABLE 64.—SUMMARY OF DATA OBTAINED FROM GAGINGS OF DRY WEATHER SEWAGE FLOW, MADE IN 1910, PHILADELPHIA, PA.

Name of area	Character	Area in acres		Population, census 1910		Average discharge per 24 hours, gallons	
		Total	Settled 1910	Total	Per settled acre	Per settled acre	Per capita
Thomas Run....	Residential, mostly pairs of two and three-story houses.	320	240	15,012	62.5	14,200	227
		426	337	21,677	64.0	9,860	153
		1,094	627	36,336	58.0	9,850	170
Pine St.........	Residential, mostly solid four to six-story houses.	160	156	15,152	97.0	26,300	271
Shunk Street....	Residential, mostly rows of two and three-story houses.	208	208	25,754	123.0	10,500	85
		331	331	37,916	114.0	10,600	93
Lombard St.....	Residential, tenements, and hotels.	147	145	16,363	113.0	34,750	308
York St	Residential and manufacturing.	358	354	33,340	94.0	36,000	383
		58	36	The population contributing sewage is not shown by the census figures		99,250	
Market Street...	Commercial.	123	80[1]			92,800	

[1] This area is practically entirely built up. The settled area is "total" minus street area. None of the other areas is similar to it and the "settled area" includes the street area in each case.

direction of the authors by the Sewer Division of the Department of Public Service of that city, H. M. Waite, Chief Engineer. The gagings were made by E. J. Miner, assistant engineer, under the immediate direction of H. S. Morse, sewerage engineer.

Two districts consisting largely, but not exclusively, of residential property, were studied, the results being given in Table 65. Conventional curves, three of which are shown in Fig. 70, were plotted as representing typical flows, the resulting figures for all the districts being given

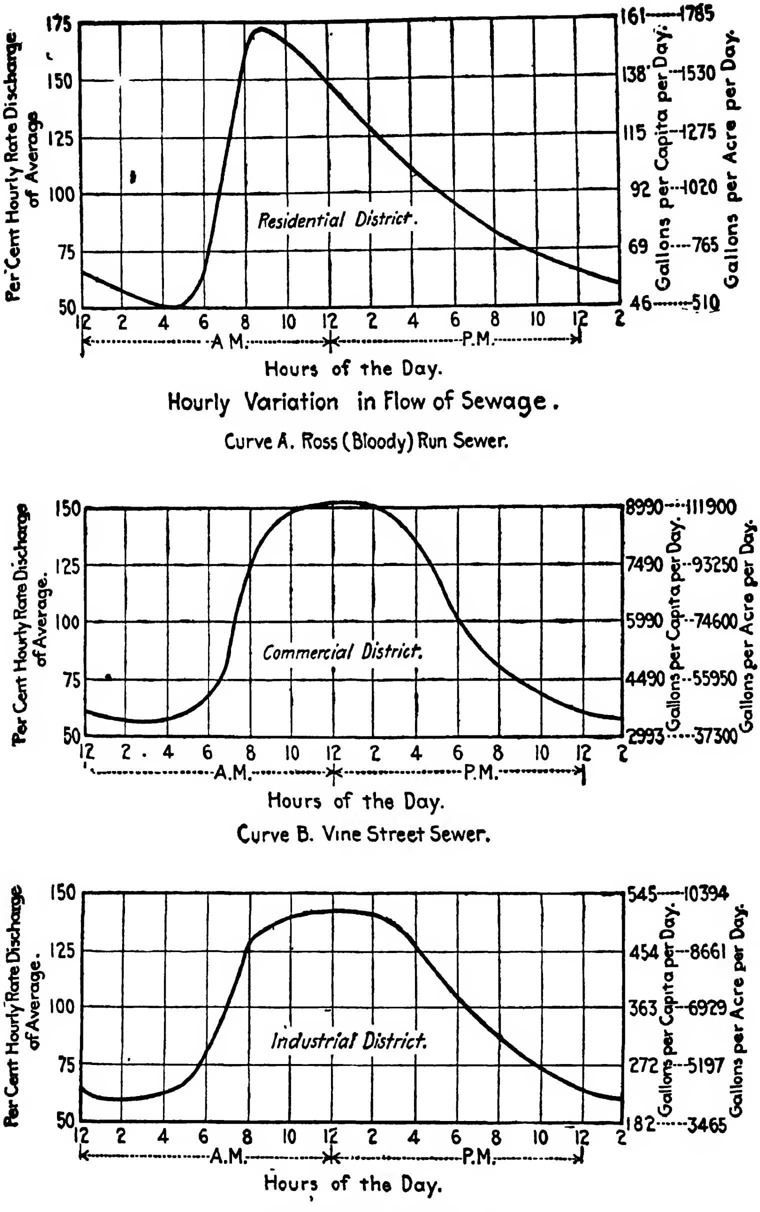


FIG. 70.—Hourly variations in flow of sewage, Cincinnati.

TABLE 65.—AVERAGE FLOW OF SEWAGE FROM RESIDENTIAL DISTRICTS, CINCINNATI, OHIO, 1912

Sewer district	Area in acres	Population		Sewage flow from actual gagings				No. of gagings covering 24-hour day	Dates of gagings
				Gals. per acre per day		Gals. per capita per day			
		Total	Density	Avg.	Max.[1]	Avg.	Max.[1]		
Ross Run.... ...	1,617	17,912	11.1	1,028	2,820	93	254	2	Dec. 3, 4.
Mitchell Ave. ...	1,650	14,781	9.0	687	1,440	77	160	5	Nov. 19, 20, 21, 22, 23
Totals and averages..........	3,267	32,693	......	857	2,130	85	207		

[1] Maximum during gaging period.

in Table 66. In spite of the fact that these are large districts, it will be seen that the maximum rate of flow must be expected to reach from 160 to 170 per cent of the average for 24 hours, and in the Ross Run district the maximum gaged flow was equivalent to 254 gal. per capita per day.

Mercantile Districts.—The allowance for used water from a mercantile district is more difficult to estimate than that from a residential district. If the estimate is to be made in connection with the design of intercepters, pumping stations, or treatment works, in which cases the district sewers have usually been built, it is highly desirable to gage the flow in the sewers and then allow for such increase due to future development as may appear to be warranted. It may also be possible to make a water supply census, being careful to ascertain the quantities of any private supplies.

Some assistance may be derived from data of the Cincinnati gagings. The commercial district of Cincinnati is located on the plateau at the top of the right bank of Ohio River, and is traversed by parallel streets running north and south in which the main sewers were built many years ago. Each sewer served a relatively small area and discharged directly into the Ohio River. Vine Street is most highly developed, and perhaps, may be said to be the center of business activities. The district gradually becomes less and less highly developed toward the east and west, grading into residential districts thickly built up with apartment houses, as reflected by the density of population shown by Table 68.

The sewers in this district are generally above the water table and infiltration in material quantities may be expected only at seasons when ground water is unusually high. Measurements of depth of flow at the

TABLE 66.—RATE OF SEWAGE FLOW FOR EACH HOUR OF THE DAY, IN PERCENTAGES OF THE AVERAGE RATE, CINCINNATI, OHIO

Time	Residential districts		Commercial districts								Industrial district
	Ross Run	Mitchell Ave.	Sycamore St.	Main St.	Walnut St.	Vine St.	Race St.	Elm St.	Plum St.	Central Ave.	Marshall Ave.
1 A. M.	63	75	33	71	55	60	42	38	77	72	59
2	58	73	31	71	52	57	41	36	77	72	60
3	53	71	33	71	50	57	41	34	77	71	61
4	51	70	35	71	50	58	41	34	80	71	63
5	52	72	40	71	55	60	41	38	84	72	66
6	64	80	53	75	66	66	48	49	89	76	81
7	112	105	74	96	92	85	67	97	100	93	105
8	162	153	126	139	144	128	145	151	118	113	129
9	171	162	171	147	156	141	174	170	130	126	134
10	167	156	190	147	158	148	175	177	134	136	138
11	157	138	191	140	154	150	174	180	137	139	140
12 M.	148	123	190	135	150	152	173	178	137	139	141
1 P. M.	139	114	185	128	144	152	171	174	132	137	141
2	128	108	180	120	136	151	169	165	125	136	141
3	118	105	172	116	128	147	168	153	118	131	136
4	109	101	159	109	123	136	165	140	107	128	126
5	102	98	136	102	113	118	155	124	100	122	116
6	94	95	107	98	105	99	89	106	96	108	105
7	88	92	72	91	97	89	71	90	86	89	94
8	82	89	56	88	91	81	62	75	84	80	86
9	79	86	48	85	82	74	53	62	80	72	80
10	74	84	42	81	75	68	47	52	77	72	73
11	70	80	39	75	66	63	42	45	77	72	68
12 P. M.	66	77	36	72	61	60	42	40	77	72	63

These figures have been computed from the average or conventional curves for the several sewer districts. Sunday flow has not been included in preparing these conventional curves.

point of observation were made at frequent intervals throughout the 24 hr. and extended over one or two days in each case. The average and maximum rates of flow per acre and per capita per day are given in Table 67.

The rates of flow vary greatly from hour to hour, due to the way in which the water is used. The districts are so small that these fluctuations are not smoothed out as they would be in large districts or in an

intercepting sewer. From the gagings conventional curves were plotted as representing what might be termed typical rates of flow, the resulting fluctuations being given in Table 66. The curve for Vine Street is shown as Curve B, Fig. 70.

TABLE 67.—AVERAGE FLOW OF SEWAGE FROM COMMERCIAL DISTRICTS, CINCINNATI, OHIO, 1912

Sewer district	Area in acres	Population		Sewage flow from actual gagings				No. of gagings covering 24-hour day	Dates of gagings
				Gals. per acre per day		Gals. per capita per day			
		Total	Density	Average	Maximum[1]	Average	Maximum[1]		
Sycamore St ..	27.8	1,702	61.2	25,800	76,300	421	1,245	1	Oct. 30, 31.
Main St. ...	18.5	487	26 3	37,750	88,100	1,435	3,350	2	Oct. 29, 30.
Walnut St.....	29 2	380	13 0	60,000	135,000	4,610	10,360	1	Nov. 2.
Vine St	23.6	294	12.5	72,000	139,000	5,780	11,160	1	Nov. 5
Race St.......	37.7	655	17.4	48,250	89,400	2,777	5,150	2	Nov. 15, 16
Elm St........	32 5	1,226	37.7	40,800	81,250	1,080	2 150	2	Nov. 8, 9
Plum St	29.5	1,231	41.7	14,700	35,300	352	845	1	Nov. 12
Central Ave...	28.4	1,579	55.6	22,050	38,400	396	690	2	Nov. 12, 13
Totals or averages........	227.2	7,554	33.2	40,169	85,344	2,106	4,369		

[1] Maximum during gaging period.

Industrial Districts.—The amount of industrial wastes not originating in the public water supply is subject to wide variations in different cities and is a matter for individual study in any particular case. The amount of such wastes may be large in some cities, and may even exceed the volume of house sewage. The amounts of these wastes have been investigated or estimated in a number of cities and the results of a few of such studies are given in Table 68.

TABLE 68.—ESTIMATES OF INDUSTRIAL WASTES ENTERING SEWERS

City	Gal. per capita per day	Date of estimate
Milwaukee, Wis......................	57	1911
Fitchburg, Mass......................	81 (max.)	1911
Passaic Valley Sewer..................	38 (max.)	1908
Louisville, Ky.........................	57	1906
Paterson, N. J.........................	18 (max.)	1906
Providence, R. I.......................	42 (max.)	
Mass. Neponset Valley intercepter.......	25 (max.)	1895
Cincinnati, Ohio......................	50	1912

In Table 69 the results of gaging one industrial district are given. This district contained both residential and industrial areas but is typical of many sewer districts in industrial centers. These gagings extended over 3 days and the maximum rate of flow found was over 13,000 gal. per acre, equivalent to over 700 gal. per capita. The hourly fluctuations to be expected in this district, taken from a smooth curve based on the gagings, are given in Table 66 and the curve is shown as Curve *C* in Fig. 70.

TABLE 69.—AVERAGE FLOW OF SEWAGE FROM AN INDUSTRIAL DISTRICT, CINCINNATI, OHIO, 1912

Sewer district	Area in acres	Population		Sewage flow from actual gagings				No. of gagings covering 24-hour day	Dates of gagings
				Gals. per acre per day		Gals. per capita per day			
		Total	Density	Avg.	Max.[1]	Avg.	Max[1]		
Marshall Ave....	294	5,611	19.1	6,787	13,485	356	708	3	Nov. 26, 27, 30

[1] Maximum during gaging period.

TOTAL QUANTITY OF SEWAGE

Having given consideration to the population, area, and average and maximum rates of flow to be expected in residential, mercantile, industrial, and park districts, it is next necessary to combine the different elements to arrive at an estimate of quantity of sewage for which provision should be made for the entire community, or the portion of it which may be served by a trunk or intercepting sewer. It will simplify the explanation of the method of estimating and serve to summarize the whole discussion, if an illustration from actual practice is given. For this purpose the studies made in Cincinnati, Ohio, in 1913, already alluded to, may be taken. Under the conditions, three main intercepter districts were decided upon and designated as the Duck Creek, Ohio River, and Mill Creek districts from the names of the water courses along which the intercepters are to be constructed.

Having first studied the local conditions and estimated the probable growth of the city as a whole, both in population and area, during the assumed "economic period of design," consideration was given to the distribution of population and area among the several sewer districts. The respective areas, as dictated by topography, were indicated upon maps and measured with the planimeter. A large map was then prepared upon which were indicated the outlines of the residential, mercantile and industrial areas and parks, railroad yards and cemeteries. The

TABLE 70.—POPULATIONS AND AREAS OF CINCINNATI SEWER DISTRICTS AS OF 1912 AND 1950 OHIO RIVER DISTRICT
Part of table to illustrate form and method

Number	Sewer districts, name	As of 1912			As of 1950								
					Area in acres								
		Area inside city (acres)	Population	Density, persons per acre[1]	Total area of sewer district	Parks and cemeteries	Railroad yards	Industrial area	Mercantile area	Residential area	Total area excluding parks, cemeteries, R. R. yards	Density persons per acre[1]	Population
50	Waldon St.	100.8	1,168	11.6	100.8	.....	.....	22.0	.....	78.8	100.8	15.0	1,510
51	Eggleston Ave.	1,509.8	50,435	39.5	1,509.8	203.0	29.2	271.8	161.0	844.8	1,277.6	49.3	63,000
52	Butler St.	12.3	400	32.5	12.3	.....	.....	12.3	.....	....	12.3	39.8	490
53	Pike St.	10.5	446	42.5	10.5	.....	.....	10.5	.....	....	10.5	40.0	420
54	Lawrence St.	15.1	368	24.4	15.1	.....	.....	15.1	.....	....	15.1	39.7	600
55	Ludlow St.	14.4	928	64.5	14.4	.....	.....	14.4	.....	....	14.4	65.3	940
56	Broadway	26.0	1,959	75.3	26.0	.....	.....	12.0	14.0	....	26.0	80.0	2,080
57	Sycamore St.	37.9	1,702	44.9	37.9	.....	.....	8.0	29.9	....	37.9	60.0	2,270
58	Main St.	28.4	540	19.0	28.4	.....	.....	5.0	23.4	....	28.4	25.0	710
59	Walnut St.	36.9	421	11.4	36.9	.....	.....	6.0	30.9	....	36.9	10.0	370
60	Vine St.	33.7	308	9.1	33.7	.....	.....	7.0	26.7	....	33.7	10.1	340
	Totals of districts 40 to 80 inclusive	15,614.3	238,794	16.0	17,266.5	540.2	118.0	2,092.0	721.2	13,795.1	16,608.3	18.3	303,826

[1] In computing density of population, areas of parks, cemeteries, and railroad yards are deducted from total area of sewer district.

TABLE 71.—ESTIMATED QUANTITY OF CINCINNATI SEWAGE TO BE PROVIDED FOR AT MAXIMUM RATE OF FLOW AS OF 1950
OHIO RIVER DISTRICT
Part of table to illustrate form and method

Number	Sewer districts, name	Estimated future population	Area in acres			Million gallons per day				Estimated total quantity		Cumulative quantities	
			Total area	Industrial area	Mercantile area	Water supply reaching sewers 135 g.c.d.	Ground water 750 g.a.d.	Industrial sewage 9,000 g.a.d.	Mercantile sewage 40,000 g.a.d.	m.g.d.	c.f.s.	m.g.d.	c.f.s.
50	Waldon St.	1,510	100.8	22.0		0.204	0.076	0.198		0.478	0.7	14.061	21.7
51	Eggleston Ave.	63,000	1,509.8	271.8	161.0	8.505	1.132	2.446	6 40	18.523	28.7	32.584	50.4
52	Butler St.	490	12.3	12.3	.....	0.066	0.009	0.111	....	0.186	0.3	32.770	50.7
53	Pike St.	420	10.5	10.5	...	0.057	0.008	0.094	....	0.159	0.2	32.929	51.0
54	Lawrence St.	600	15.1	15.1	...	0.081	0.011	0.136	..	0.228	0.3	33.157	51.3
55	Ludlow St.	940	14.4	14.4	..	0.127	0.011	0.130	...	0.268	0.4	33.425	51.7
56	Broadway	2,080	26.0	12.0	14.0	0.281	0.020	0.108	0.560	0.969	1.5	34.394	53.2
57	Sycamore St.	2,270	37.9	8.0	29.9	0.306	0.028	0.072	1.196	1.602	2.5	35.996	55.7
58	Main St.	710	28.4	5.0	23.4	0.096	0.021	0.045	0.936	1.098	1.7	37.094	57.4
59	Walnut St.	370	36.9	6.0	30.9	0.050	0.028	0.054	1.236	1.368	2.1	38.462	59.5
60	Vine St.	340	33.7	7.0	26.7	0.046	0.025	0.063	1.068	1.202	1.9	39.664	61.4
	Totals of districts 40 to 80 inclusive	303,826	17,266.5	2,092.0	721.2	41.017	12.950	18.828	28.848	101.643	157.2		

In this table, g.c.d. is an abbreviation of gallons per capita daily: g.a.d. is an abbreviation of gallons per acre daily: m.g.d. stands for million gallons daily, and c.f.s. for cubic feet per second.

portions of each coming within each sewer district were measured and tabulated as in Table 70, together with the estimated future population.

Consideration was next given to the quantity of sewage for which provision should be made, the units of maximum rate of flow in intercepters adopted after a study of local conditions and all data available being given in Table 72.

TABLE 72.—UNIT QUANTITIES OF FLOW IN INTERCEPTERS ASSUMED FOR CINCINNATI, OHIO

Residential areas:	
Sewage	135 gal. per capita per day
Ground water	750 gal. per acre per day
Mercantile areas:	
Sewage (resident population)	135 gal. per capita per day
Additional allowance for character of development	40,000 gal. per acre per day
Ground water	750 gal. per acre per day
Industrial areas:	
Sewage (resident population)	135 gal. per capita per day
Industrial wastes	9,000 gal. per acre per day
Ground water	750 gal. per acre per day
Parks, railroad yards, and cemeteries:	
Ground water	750 gal. per acre per day

The results obtained by the computations are illustrated by Table 71 and are summarized for the whole city in Table 73.

A few comments may serve to explain some of the reasons for the units adopted. The rates in all cases are the highest anticipated at times when it will be necessary to intercept the sewage or ultimately to treat it. They also are influenced by the smoothing-out effect of the differences in time of entrance of sewage from lateral sewers into trunk sewers and from trunk sewers into intercepters. The ground-water allowance is very low, because of the enormous area and sparse population anticipated, and because of the topography, which assures that a large portion of the sewerage system will be above the water table during the drier portions of the year. These conditions appeared to warrant the adoption of a ground-water factor much lower than the authors have dared to use in a number of other cities.

The proportions of the several classifications of flow in the several districts and for the entire city are given in Table 74. From these data it will be seen that there are great differences in the allowances for the several intercepters, more than twice as great a flow per acre being provided for in the Ohio River intercepter as in either of the others.

Provision for Storm Water.—There is a general impression that it is wise, in intercepting sewers, to provide for a small quantity of storm water, expressed often as being sufficient for the "first flushings" of street

TABLE 73.—ESTIMATED TOTAL QUANTITY OF SEWAGE TO BE PROVIDED FOR AT MAXIMUM RATE OF FLOW IN THREE INTERCEPTER DISTRICTS, CINCINNATI, OHIO, AS OF 1950

Drainage district	Estimated future population	Area in acres			Million gallons per day				Estimated total quantity		Quantity in units of	
		Total area	Industrial area	Commercial area	Water supply reaching sewers 135 g.c.d.	Ground water 750 g.a.d.	Industrial sewage 9,000 g.a.d.	Commercial sewage 40,000 g.a.d.	m.g.d.	c.f.s.	Gallons per acre per day	Gallons per capita per day
Mill Creek	308,664	52,740.1	4,863.3	.....	41.670	39.555	43.770	......	124.995	193.4	2,370	405
Ohio River	303,826	17,266.5	2,092.0	721.2	41.017	12.950	18.828	28.848	101.643	157.2	5,885	334
Duck Creek	99,320	12,690.9	1,228.5	.....	13.408	9.518	11.056	......	33.982	52.6	2,680	342
Total	711,810	82,697.5	8,183.8	721.2	96.095	62.023	73.654	28.848	260.620	403.2	3,150	366

surfaces and sewers. This impression is based upon the assumption that there are accumulations of sewage sludge in the sewers and quantities of filth on the streets which will immediately be flushed into the intercepting sewers with the first runoff due to rain. In some sewers laid on very flat grades, or where sewers have settled or have been built with depressions in them, there may be such deposits, but where sewers are laid on grades which give satisfactory velocities, such deposits are believed to be exceptional. Where deposits occur they are generally found to consist largely of sand and other heavy detritus which will be carried along only by relatively high velocities.

TABLE 74.—ESTIMATED UNIT QUANTITIES OF SEWAGE TO BE PROVIDED FOR AT A MAXIMUM RATE OF FLOW IN THREE MAIN DRAINAGE DISTRICTS, AS OF 1950, CINCINNATI, OHIO

	Duck Creek intercepter			Ohio River intercepter			Mill Creek intercepter			Whole city		
	Gal. per acre per day[1]	Gal. per capita per day	Per cent	Gal. per acre per day[1]	Gal. per capita per day	Per cent	Gal. per acre per day[1]	Gal. per capita per day	Per cent	Gal. per acre per day[1]	Gal. per capita per day	Per cent
Sewage from resident population	1,057	135.0	39.5	2,375	135.0	40.4	790	135	33.4	1,162	135.0	36.9
Additional allowance, mercantile areas				1,670	95.0	28.4	. . .	. .	. .	349	40.6	11.1
Additional allowance industrial areas	873	111.2	32.5	1,090	62.0	18.5	830	142	35.0	890	103.3	28.2
Ground water	750	95.8	28.0	750	42.0	12.7	750	128	31.6	750	87.1	23.8
Totals	2,680	342.0	100.0	5,885	334.0	100.0	2,370	405	100.0	3,150	366.0	100.0

[1] Total area of district.

Industrial sewage, based upon 9,000 gal. per acre of industrial area; mercantile sewage, upon 40,000 gal. per acre of mercantile area; ground water, upon 750 gal. per acre of total area; domestic sewage, upon 135 gal. per capita.

Intercepters are fed by trunk sewers serving rather large districts. Considerable time is required to flush the major part of the system to the intercepter, during which a large flow is likely to reach it from the nearer portions. Unless considerable surplus capacity is provided, the intercepter will often be running full before the flushings from much of the tributary area can reach it. Therefore, too much stress should not be laid on ability to care for the "first flushings," although, as ordinarily designed, they can accomplish something in this direction.

Assuming that the average daily flow of sewage is 100 gal. per capita and that the capacity of the intercepter is 300 gal. per capita, there will be a surplus capacity available for "first flushings" equivalent to twice the average flow of sewage, if such flushings come at a time when the

flow is at the average rate. Except during storms, the maximum rates of flow generally occur in the spring when ground water is high, and at other times there will always be some surplus capacity. Furthermore, as intercepters are built for anticipated future requirements, there will be a considerable excess capacity during the earlier years, although this should not usually be counted upon to care for storm water, for it is a gradually diminishing allowance accompanied by a gradually increasing need, if need there should be.

Under the foregoing conditions the sewage will be diluted to three times its normal flow. Furthermore, consideration must be given to the excess of water used in this country and to the quantity of ground water which leaks into the sewers. With these eliminated, the quantity of sewage would be comparable with that obtained in Europe. Taking all these conditions into account, it is evident that the dilution approaches the standard of the Royal Commission on the Disposal of Sewage, which is six times the dry-weather flow.

In view of these conditions, it was not deemed wise to provide capacity in the Cincinnati intercepters for storm water in excess of that which can be carried when the flow of sewage is less than the maximum rates assumed as already described; in other words, no special allowance for storm water was made.

Caution.—The foregoing outline of a rational method of estimating the quantity of sewage to be provided for applies to the design of intercepters and large trunk sewers, and the units adopted are for maximum rates of flow when the sewers are running full.

Average Rates of Flow.—The average rates of flow upon which estimates of cost of pumping and treatment may be based are much below the maximum rates, and, from the data available, appear to range in a general way between 100 and 125 gal. per capita per day for the larger cities. For small towns, average rates appear to range from about 25 to 60 gal. per capita per day.

Relation between Average and Maximum Rates of Sewage Flow.—Another somewhat empirical method of arriving at the rate of sewage flow for which sewers are to be designed is first to estimate the average rate of flow, which may be approximated from the water consumption and the allowance for ground water, and then to apply a factor for the ratio between maximum and average rates of flow. This ratio will not be a constant, but will vary with the magnitude of the flow, as well as with other conditions.

Figure 71 is a curve showing the relation between average rate of sewage flow and rate to be used in design, which was prepared by the Maryland State Department of Health, using actual measurements of flow in various cities.[1]

[1] *Municipal Jour.*, 1915; **38**, 843.

TABLE 75.—BASIC QUANTITIES PER DAY USED IN FIXING THE SIZE OF INTERCEPTING SEWERS IN VARIOUS CITIES

City	Ultimate		Average density of population	Estimate water consumption, gallons per capita	Average flow gallons per capita			Storm water			Maximum rate, dry-weather flow of sewage, gallons per capita	Total provision		Date of	
	Area in acres	Population			Domestic sewage	Industrial wastes	Ground water	Inch	Gallons per capita	Gallons per acre		Gallons per capita	Gallons per acre	Design	Ultimate estimate
Chicago.															
North Lake front; majority report	3,020	120,800	40.0					6.0	4,073	162,910	180	4,282	171,270	1897	
Minority report	3,020	120,800	40.0					2.0	1,360	54,400	180	1,540	61,600	1897	
South Lake front; majority report	7,401		[3]					[4]			180		166,890	1897	
Minority report	7,401		[3]					[5]						1897	
Boston, Mass.[1]	9,600	600,000	62½	75	75			0.24	108	6,730	150	258	16,125	1877	
Massachusetts North Metropolitan intercepter	46,000	513,000	11.1									239[6]	2,670	1889	1930
Metropolitan Chas. River intercepter	26,900	157,000	5.9									224	1,316	1889	1930
Metropolitan Neponset River valley intercepter	21,864	213,316	9.8			25 max.						223	2,190	1895	1930
Metropolitan high-level sewer	64,600	986,000	15 2	140			24 max.				200	300	4,580	1899	1940
Baltimore		1,000,000										300[7]		1906	1925 to 35
Providence	11,351	337,000	29.7		60	42 max.		0.24	218	6,470	145	363	10,780		
Paterson[2]		300,000		75	75	18 max.	14					250		1906	1938
Louisville	7,829	251,800	32.5		213[1]	72[10]	90[10]					375	12,100	1907	
Milwaukee	15,433	562,010	36.4	100				0.24	178	6,480		419	15,240	1889	1935
Milwaukee	35,000	850,000	22.8	105 to 125	70 to 85[9]	57.2	75 to 81				248 to 271	350	8,500	1910	1950
Milwaukee	37,400	862,000	23		50	36	59			32,400	305			1915	1950
Passaic Valley sewer project	61,827	1,649,440	27.1		100	38 max.	29 max.					241[8]	6,422[8]	1908	1940
New Bedford	8,808	220,695	25.1								125	400	10,000	1911	1940
Syracuse	10,933	410,000	37.5	90							125	375	14,063	1895	revised 1907
Fitchburg, Mass.	5,858	82,160	12.3		150	81	139					432	5,180	1911	1940
Fitchburg, Mass.	8,134	87,200	10.7	100	75	183 max.	115 max.					450	4,800	1912	1940
Fort Wayne, Ind.	12,160	150,000	12.3	100	150		160				150	350	4,310	1911	1950

Cleveland	...	1,000,000	...	...	150	...	...	...	...	...	200	400	...	1895	
	...	1,700,000	...	...	...	...	...	...	...	...	...	400	...	1914	
Cincinnati	82,698	711,810	...	...	135	...	...	...	...	...	...	366	3,150	1913	1950
Lynn, Mass.	...	110,000	40	...	...	...	...	...	...	...	135	810	32,400	1884	
Chicago, Sanitary District:															
Calumet	...	...	...	...	100	67	50	...	...	...	282	350	...	...	1960
North Side towns	...	...	...	...	90	7	20	...	...	...	201	482	...	...	1960
North Side (city)	...	...	...	...	112	12	36	...	...	...	208	323	...	...	1960
West Side	...	...	...	...	148	74	28	...	...	...	342	500	...	...	1960
Southwest Side	...	...	...	...	140	30	30	...	...	...	252	400	...	...	1960

Table 75 was prepared from the following sources:

Chicago.—*Report* of Chicago Commissioner, 1897.

Boston.—Main Drainage Works of Boston and its Metropolitan Sewerage District (p. 8), published under authority of Metropolitan Sewerage Commissioners, 1899.

Massachusetts, North Metropolitan Intercepter.—*Report* of Massachusetts State Board of Health, Drainage of Mystic and Charles River Valleys, 1890.

Massachusetts, Charles River Valley Intercepter.—*Idem.*

Massachusetts, Neponset River Valley Intercepter.—*Report* Metropolitan Sewerage Commissioners, 1895, (p. 38).

Massachusetts Metropolitan High-level Sewer.—*Report* Metropolitan Sewerage Commissioners High-level Gravity Sewer (1/1899), (p. 44).

Baltimore.—Baltimore Sewerage Commissioner, 1906 (p. 23).

Providence.—Furnished by City Engineer.

Paterson.—*Report*, Joint Commission on Sewage Disposal, 1908, (p. 111).

Louisville.—Furnished by H. P. Eddy. Also *Report* Commissioners of Sewerage, 1921 (p. 108).

Milwaukee, 1889.—*Report* of Commission of Engineers.

Milwaukee (1910).—*Report* of Commission of Engineers.

Milwaukee, 1915; *Annual Report* Sewerage Commission.

Passaic Valley Sewer Project.—*Report* Passaic Valley Sewerage Commissioners, 1908 (p. 7).

New Bedford.—*Report* of Metcalf and Eddy, 1911.

Fitchburg.—*Report* of Metcalf and Eddy, 1911.

Syracuse.—From *Eng. News* (July 13, 1911, p. 38).

Fort Wayne, Ind.—*Report* of Metcalf and Eddy, 1911.

Cleveland.—*Report* of Hering, Benzenberg, and Fitzgerald, 1886. *Report* of Department of Public Service, 1914.

Cincinnati.—*Report* of H. S. Morse and H. P. Eddy, 1913.

Lynn.—*Report* of Rudolph Hering, 1884.

Chicago, Sanitary District.—*Report* of Engineering Board of Review, 1925.

[1] Main drainage figures are with sewers half full. The slope and size of the outlet section of intercepter indicates a capacity, flowing full, of 407 gal. per capita. [2] See Paterson report for explanation of figures. Area taken from census report and may vary somewhat from actual areas used in design. [3] Density, 60 in district tributary to 12th and 22nd St. sewers; 70 in district tributary to 35th St. sewers; 60 in district tributary to 41st and 45th St. sewers. [4] Storm water, 0.273 in. per hour, area north of 39th St.; 0.25 in. per hour, area tributary to 41st and 45th St. sewers; 0.22 in. per hour, area tributary to 51st St. sewer; 0.16 in. per hour, area tributary to 56th St. sewer. [5] Storm water, 0.273 in. per hour 12th and 22nd St. sewer; 0.15 in. per hour 35th St. sewer; 0.0833 in. per hour 39th St. sewer; 0.0625 in. per hour 63rd St. sewer. [6] 224 gal. in districts provided with separate system of sewers; 261.8 gal. in districts provided with combined system of sewers. [7] Pumping plants and parts easily duplicated 150 gal. per day. Disposal plant 75 gal. per capita per day. [8] Passaic Valley figures are for sewer flowing full. Figures in report assume maximum flow with sewer ¾ full. [9] Water supply reaching sewers. [10] Maximum.

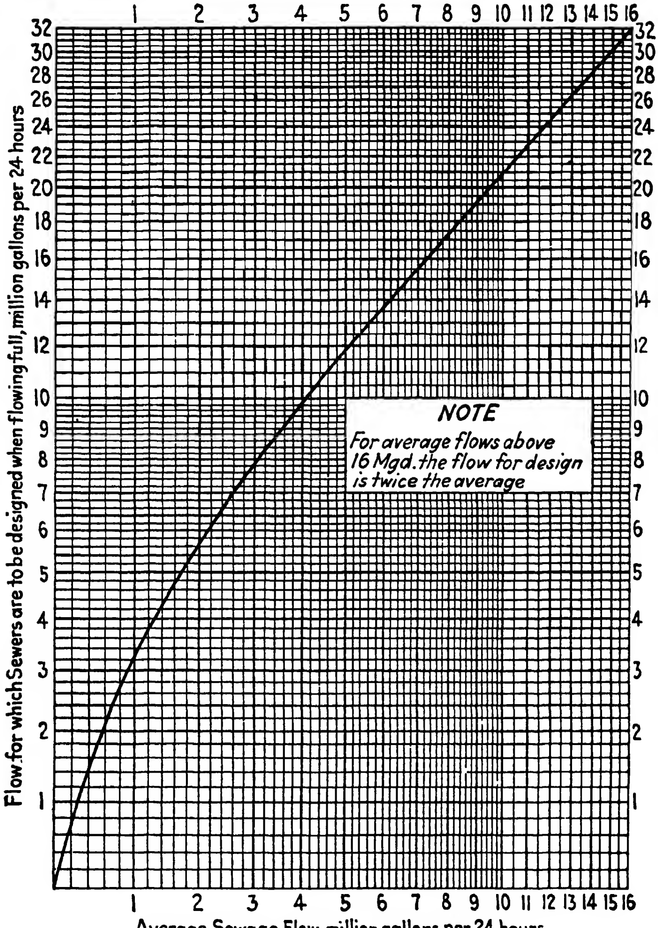


FIG. 71.—Diagram for converting average sewage flow to flow for design in sewers of separate system. (*Maryland State Department of Health, January, 1914.*)

BASES OF DESIGN OF EXISTING INTERCEPTERS

The allowances made by engineers in the design of a number of existing intercepting sewers are given in Table 75. Some of the older intercepters, designed on a basis of less than 300 gal. per capita, now appear to be inadequate for the service they will ultimately be required to render and more recent designs are more liberal.

CHAPTER VI

SEPARATE SEWERS

Definitions.—*A separate sewer* is a sewer intended to receive domestic sewage and industrial wastes without admixture of surface or storm water.

A combined sewer is a sewer intended to receive domestic sewage, industrial wastes, and surface and storm water.

A drain is a conduit for carrying off storm water, surface water, and sub-soil or ground water (domestic sewage and industrial wastes being excluded).

The separate system of sewerage contemplates the ultimate provision of two distinct systems of conduits, one of separate sewers, the other of drains.

Reasons for Adopting the Separate System.—The construction of a system of separate sewers without a system of storm drains, or with only a partial one, has become common practice in small communities, and is somewhat prevalent in the larger cities. This has been due generally to economic necessity, either real or fancied. The small towns frequently consider it financially impossible to finance an adequate system of combined sewers, and it is often possible to allow storm water to flow in gutters and in natural water courses for many years after the necessity for separate sewers has become pressing. Even where combined sewers are not prohibitive in cost, it is generally easier to secure funds for the less expensive separate sewers. This often leads to the adoption of separate sewers without a thorough study and careful weighing of the comparative merits of the two systems.

In some cases, also, the discharge of sewage mingled with storm water, even during brief overflow periods of storms, is not permissible—a condition which may justify the adoption of separate sewers.

Misuse of Separate Sewers.—Annoyance and damage have resulted from the discharge of storm and ground water into separate sewers. This occurs through the connection with the sewers of roofs, street inlets, and foundation and cellar drains, and, in some cases, even small brooks. Sewers are sometimes so poorly built that they receive large quantities of ground water that should be cared for by natural channels or storm drains. Even where the sewers are most carefully built to exclude extraneous water the house connections are often so improperly made

that large quantities of ground water find access to the sewers, thus nullifying the effort and money expended in securing excellence of workmanship in their construction.

The results of such abuses have been serious in many places. In others, however, the ill effects have not yet been so important, because the improper connection of roofs and inlets and the faulty workmanship upon sewers and house connections go on gradually and considerable time may be required for their cumulative effect to become such as to cause damage and demand public attention. The effect of such abuses is destined to increase greatly as time goes on. In one case, a city has been compelled approximately to double the size of its sewage treatment plant, solely because of the admission of water to separate sewers.

The sources of this water are many, including defective building connections, street inlets, perforated manhole lids (particularly within street-car tracks), and abandoned building connections which were not sealed when buildings were burned or removed. One of the most important sources in the case cited above appears to have been wet cellars from which water was drained to the separate sewers through the cleanouts in cast-iron soil pipes laid below cellar floors. There are many buildings located on side hills in some of which there is ledge. During and after rains, large quantities of ground water enter the cellars, and the owners, or tenants, remove the caps from cleanouts and allow the water to drain out. In some cases, the cleanout caps have been left out continuously.

Probably few systems of separate sewers have been so designed that they can receive the roof water from more than about 1 per cent of the tributary houses without being surcharged at times of intense downpours after the districts they are to serve shall have been built up to the extent contemplated in the design. If there are also other sources of admission of water, such as street inlets and perforated manhole lids, the proportion will be correspondingly less.

In some places, the sewers have been surcharged to such an extent that sewage has flowed back through house connections into the cellars, while in others it has escaped through manhole covers into the streets. Just how far such conditions can be tolerated, without justifying the statement that the separate system in any case is a failure, is a debatable question.

DESIGN OF SEPARATE SEWERS

Basic Data.—The accumulation of the basic data for the design of separate sewers may be illustrated by an example, utilizing the data contained in a report to the Commissioners of Sewerage of Louisville, Ky., by J. B. F. Breed and Metcalf and Eddy, dated August 31, 1921.

The flow in separate sewers consists of the discharge of sewage from residences and other buildings; industrial wastes; ground water which finds its way into the sewers through porous or cracked pipe, defective joints, or porous masonry; storm water from roofs and gutter inlets which are accidentally or surreptitiously connected with the sewer; and leakage around lids or through perforated lids of manholes in the streets.

It is obviously impossible to predict with accuracy the densities of the population which will reside upon different portions of the tributary area a number of years hence. It is, therefore, wise to allow some excess capacity to provide for unexpected densities which may be found in restricted areas, in excess of the average over a large tributary area. Such provision should also be adequate for other exceptional conditions, such as the establishment of a schoolhouse, a laundry, or a large apartment house. Perhaps the most important exceptional condition for which provision must be made is the occasional connection of roofs of buildings and gutter inlets. Such connections, of course, should be prohibited, but experience in Louisville and in many other cities has demonstrated that such connections are occasionally made through

TABLE 76.—DENSITIES OF POPULATION IN BEARGRASS INTERCEPTER DISTRICT, 1920, LOUISVILLE, KY.

Drainage district	Cumulative quantities	
	Area, acres	Population, persons per acre
Green Street	118	58.3
Broadway West	589	49.6
Dry Run	1,360	39.1
Phoenix Hill	1,395	38.8
Brent Street	1,581	37.2
Underhill Street	1,618	36.8
St. Catherine St. / Dupuy Street	1,660	36.3
Southeastern	1,937	33.9
Snead's Brook	2,358	31.0
Ellison Avenue	2,445	30.6
Webster Street	2,550	29.9
Cooper Street	2,607	29.4
Castlewood	2,842	28.0
Northeastern	3,673	24.0
Middle Fork	4,525	21.3

accident or otherwise. On the other hand, the construction of all sub-main and lateral sewers of adequate capacity for grossly abnormal development, such as that of an extensive industrial district in which large quantities of liquid wastes are produced, would entail an expense which cannot be justified in the localities under consideration.. In case of such grossly abnormal development the city must adopt radical measures, such as providing relief sewers for the districts so developed.

Study of the conditions surrounding those portions of the city which are likely to be provided with separate sewers leads to the conclusion that they will be developed in such a manner that their average ultimate population will not exceed 25 persons per acre.

The former allowance for flow in separate sewers to be built in the Northeastern and Middle Fork Districts was 200 gal. per capita per day for an ultimate population of 25 persons per acre, to this being added an allowance for ground water infiltration of 1,960 gal. per acre per day. More recent experience with such sewers in other cities has led to a restudy of this subject with the results hereinafter described.

It is not to be expected that the districts in which it is proposed to instal the separate system will be as densely populated as the Beargrass Intercepter District, which now has an average density of population of about 21 persons per acre, and 40 years hence will undoubtedly be much more densely populated.

The curve of "Densities of Population in the Beargrass Intercepter District, 1920," however, has been of assistance in making allowance for the exceptional conditions mentioned. With this as a guide, another curve has been prepared and used, not as an estimate of probable population, but merely as a means of arriving at a reasonable allowance for maximum rate of flow in separate sewers in the district under consideration. A few illustrations of density of population taken from this curve are given in Table 77.

TABLE 77.—ALLOWANCES FOR DENSITY OF POPULATION IN DISTRICTS SERVED BY SEPARATE SYSTEMS OF SEWERS

Extent of district, acres	Density of population, persons per acre
10	100
100	63
250	52
500	46
1,000	40
2,500	33
5,000	29
10,000	25

The allowance for maximum rate of flow in separate sewers has been based upon the following general data:

Maximum possible density of population for areas of 10 acres or less..........	100 persons per acre
Maximum possible density of population for areas between 10 and 1,000 acres..	100–40 persons per acre
Maximum possible density of population for areas of 1,000 acres or more......	40 persons per acre
Maximum rate of sewage discharge from buildings..........................	200 gal. per capita per day[1]
Maximum rate of ground-water infiltration..	1,960 gal. per acre per day[2]

From a combination of these general data, the allowance for maximum rate of sewage flow may be obtained, as shown by the curves in Fig. 72 and the data in Table 78.

TABLE 78.—DATA RELATING TO METHOD OF ESTIMATING MAXIMUM RATE OF SEWAGE FLOWING IN SEPARATE SEWERS, LOUISVILLE, KY.

Area of district, acres	Maximum rate of sewage flow, gallons per acre per day	Maximum allowance for sewage flow provided population should equal flat rate of 25 persons per acre, gallons per capita per day	Number of persons per acre for which provision would be made on basis of 300 gallons per capita per day	Number of roofs which could be connected with sewers without causing surcharge assuming a uniform density of population of 25 persons per acre, one in
10	21,900	875	73	40
25	18,500	740	62	42
50	16,500	660	55	45
100	14,600	585	49	45
250	12,500	500	42	47
500	11,100	445	37	54
750	10,400	415	35	63
1,000	9,960	400	33	69
2,000	9,960	400	33	59
5,000	9,960	400	33	48

It will be seen that the allowance for rate of maximum flow decreases with increase in area to 1,000 acres, beyond which the allowance is for a uniform rate per acre. In this way, provision is made for unusual development of small areas, limited industrial or mercantile development, or admission through error of a very small quantity of roof water.

Where any of these demands, or all of them, will become effective, cannot be foretold. It is unlikely that they all will be made upon any one

[1] Note that this quantity might be seriously in error for other places.

[2] Approximately 50,000 gal. per day per mile of sewers for completely sewered area.

lateral and certain that they will not all be made on all laterals. For this reason the allowance for surplus above the estimated average flow of house sewage and ground water is made to decrease to the point where an area 1,000 acres in extent is served. Beyond this point, the flow per acre is arbitrarily assumed to be uniform, and the surplus also is uniform.

One way of judging of the extent of the surplus allowed is by considering the maximum rate of discharge which could be carried away in case the population should be 25 persons per acre, a density which it is believed may be reached within the next 40 years. The allowance per capita upon this basis (column 3) is seen to vary from 875 gal. per day for areas of 10 acres or less, to 400 gal. per day for areas of 1,000 acres or more.

Or it may be helpful to consider how many persons could be served if the maximum rate be assumed at 300 gal. per day per capita, a rate

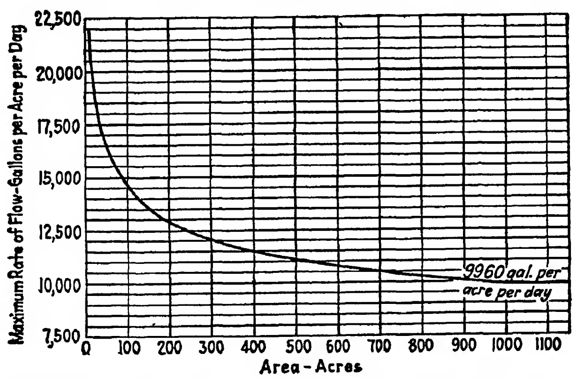


FIG. 72.—Maximum rate of flow in separate sewers, including ground water. (*Louisville, Ky.*, 1921.)

commonly used in estimating the size of intercepters in which no additional provision is made for storm water. From column 4 it will be seen that the number varies from 73 to 33 persons per acre.

Again, if there is no unusual development and the population ultimately becomes 25 persons per acre, the surplus will all be available for carrying such storm water as may accidentally be admitted to the sewers.

In the case of the 10-acre plot, the roof water from about 2½ per cent of the houses can be admitted without surcharge of the sewers. At 7½ houses per acre this is equivalent to the roof water from about two houses. In the case of 1,000 acres about 1½ per cent of the houses can be accommodated, which is equivalent to about 100 houses. The number of roofs from which storm water can be received without surcharging the sewers is estimated on the assumption that 80 per cent of the water falling upon the connected roofs will reach the sewers, that the rate of precipitation will vary, approximately according to the 15-year frequency precipita-

tion curve,[1] and that there will be a uniform velocity of 3 ft. per second in the sewers.

It is obvious that provision is not made for the full amount of excess flow which might possibly arise from all of these causes, but that such surplus capacity as may be provided will be available for whatever excess flow there may be from one or all of them up to the limit fixed. In case the maximum rate of 200 gal. per capita per day house sewage, or 1,960 gal. per acre per day ground water, is not reached, the sewers will, of course, have a still greater surplus capacity than that expressly provided for in the method of computation described.

Wherever separate sewers are provided, it is absolutely necessary that roof and street water be rigidly excluded. If this policy is not strictly adhered to and such waters are discharged into the sewers, the system will certainly become overcharged and not only fail to function properly but will actually become a source of annoyance and damage, for which the city may be held liable. The allowance herein advised for roof water is provided solely for connections accidentally made. Where any such connection comes to the knowledge of the city, it should forthwith be disconnected.

Maps and Profiles.—In the design of a system of separate sewers, a topographic map is desirable, on a scale of 100 or 200 ft. to the inch, showing contours, brooks, rivers, ponds, and streets. The contours should be sufficiently close to allow the designer to plot profiles of streets with reasonable accuracy; *i.e.*, where the surface slope is 6 per cent or less the map should show contours at 2-ft. intervals; where the surface slope is much greater than this, 5-ft. intervals will usually suffice. Summits in streets should be marked and the elevations given to tenths of a foot, as should also points of depression or "pockets."

Profiles of all or most of the sewers should be drawn after the slopes of the sewers have been tentatively determined, with the particular object of ensuring that the sewers are everywhere at a suitable depth to serve the buildings and at the same time to note whether drops in the manholes may be advantageous at certain points, or whether the design can be improved in other particulars. A profile of the main or trunk line will generally be desirable in advance of the computations.

Design of Sewers.—After compiling the basic data and determining the minimum allowable size of sewer and minimum velocity of flow in the full sewer, the procedure of design is as follows:

1. Draw lines on the topographic map to represent the sewer in each street.
2. Locate the manholes.
3. Sketch the limits of drainage areas for each lateral sewer.
4. Measure the drainage areas.

[1] Discussed in detail in the next chapter, and illustrated in Fig. 80, p. 265.

TABLE 79.—COMPUTATIONS FOR DESIGN OF SEPARATE SEWERS

Line number	Street	From manhole number	To manhole number	Length, ft.	Tributary area		Maximum rate of sewage flow				Slope of sewer, S	Diameter of sewer D, inches	Capacity when full Q, c.f.s.	Velocity when full v, feet per second	Fall in sewer, feet	Elevation of sewer invert	
					Increment, acres	Total, acres	Domestic and ground water from curve, m.g.d.	Industrial m.g.d.	Total							Upper end	Lower end
									m.g.d.	c.f.s.							
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)
1	Eloise	6	7	400	16.0	16.0	0.31	....	0.31	0.48	0.0090	8	0.88	2.5	3.60	173.75	170.15
2	Sumner Ave.	7	8	425	24.5	40.5	0 71	....	0.71	1.10	0.0065	10	1.4	2.5	2.76	169.98	167.22
3	Sumner Ave.	8	9	395	14.5	55.0	0.91	....	0.91	1.41	0.0065	10	1.4	2.5	2.57	167.22	164.65
4	Sumner Ave.	9	10	415	19.7	74.7	1.16	....	1.16	1 80	0.0045	12	1.8	2.5	1.87	164.48	162.61
5	Sumner Ave.	10	11	390	27.3	102.0	1.48	....	1.48	2.29	0.0065	12	2.3	2.9	2.54	162.61	160.07
6	Sumner Ave.	11	12	480	34.6	136.0	1.86	....	1.86	2.88	0.0035	15	3.1	2.5	1.68	159.82	158.14
7	White	12	13	365	[1]127.0	263.0	3.27	2.54	5.81	9.02	0.0025	24	9.5	3.0	0.91	157.39	156.48
8	White	13	14	390	39.2	302.2	3.66	....	6.20	9.60	0.0025	24	9.5	3.0	0.98	156.48	155.50
9	White	14	15	410	44.8	347.0	4.10	....	6.64	10.3	0.0030	24	11.0	3.3	1.23	155.50	154.27
10	White	15	16	425	27.0	374.0	4.35	....	6.89	10.7	0.0030	24	11.0	3.3	1.29	154.27	152.98
11	Portland	16	17	500	187.0	561.0	6.18	3.74	12.46	19.3	0.0023	30	19.5	4.0	1.15	152.48	151.33
12	Portland	17	18	405	29.3	590.3	6.44	....	12.72	19.7	0.0023	30	19.5	4.0	0.93	151.33	150.40
13	Portland	18	19	370	47.7	638.0	6.86	....	13.14	20.4	0.0025	30	21	4.2	0.93	150.40	149.47
14	Oakland	19	20	365	294.0	932.0	9.32	....	15.60	24.2	0.0034	30	24	4.8	1.24	149.47	148.23
15	Oakland	20	21	395	16.9	948.9	9.49	....	15.88	24.6	0.0036	30	25	5.0	1.42	148.23	146.81
16	Oakland	21	22	400	41.1	990.0	9.90	....	16.18	25	0.0036	30	25	5.0	1.44	146.81	145.37
				6,530	990.0						Total fall in sewer				26.54		28.38
											Fall due to increase in size				1.84		173.75
											Total fall				28.38		

[1] Industrial areas.

5. Plot a profile of the line which is to serve the lowest point or the controlling line as to elevation.

6. Prepare a table of computations.

Table 79 gives the computation for the design of the main sewer for a residential district which is to receive in addition the wastes from two small industrial districts.

In this computation, the basic data quoted above as derived for Louisville conditions are employed, and the assumptions are made that:

1. No sewer smaller than 8 in. in diameter will be used.

2. The minimum velocity of flow in a full sewer is to be 2.5 ft. per second.

3. Computations of flow will be made by diagram based on Kutter's formula (Figs. 21 and 22), using $n = 0.015$ for pipe and 0.013 for concrete sewers.

The various steps of the computation may advantageously be put in tabular form, as in Table 79.

Minimum Size of Pipe Sewers.—The assumption that the smallest size of sewer to be employed is 8 in. is commonly made in the authors' practice, although in some cases a minimum size of 6 in. is adopted. The adoption of a minimum size is necessary because experience has shown that some comparatively large objects such as scrubbing brushes, for instance, do sometimes get into sewers, and that stoppages resulting from them are less likely if sewers smaller than 8 in. (or perhaps 6 in.) are not used. Obviously, the smallest sewer should be at least as large as, and preferably larger than, the building connections in general use, so that articles which pass the building connections will be unlikely to stop the sewer.

Engineers are not in entire agreement upon the most advantageous size of building connections. The most common size is probably 6 in., but 5 in. and 4 in. are frequent. It is good practice to use one size for sewer connections and another size for drain connections, thus reducing the likelihood of either being connected to the wrong conduit in the street. The same result can be obtained by paint marks for identification of one set of house pipes.

Drop in Manholes.—In general, losses of head in well-constructed manholes are not of much consequence, although, under some circumstances, they may be noticeable. Years ago, when a manhole was usually a large well with a sewer entering at one side and leaving at the other, and frequently with a sump extending some distance below the invert of the sewer (for the ostensible purpose of collecting grit for removal), losses of head in manholes were considerable and drops should have been and usually were provided. At present, manholes are built with channels formed in them, connecting the ends of the entering and leaving sewers, and there is no material enlargement of the stream of

sewage until it rises above the bench, which is often at the elevation of the center of the sewers, and better at that of the crown.

Drops should always be introduced for the purpose of keeping the crowns of sewers up to the hydraulic gradient when full, although the sizes are different; other drops may be introduced to provide for maintaining the sewer at the proper depth to drain adjacent buildings or for the purpose of using up excess fall. Often, however, there is no excess

Table 80.—Analysis of Separate Sewer Design under Conditions of Minimum Flow

Line number	Total tributary area, acres	Min. rate of sewage flow				Proportion of total capacity, per cent	Corresponding proportion of		Velocity at time of minimum flow, feet per second
		Domestic and ground water from curve, m.g.d.	Industrial, m.g.d.	Total			Depth, per cent	Velocity, per cent	
				m.g.d.	c.f.s.				
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
1	16.0	0.051	....	0.051	0 079	6	17	50	1 25
2	40.5	0.099	....	0 099	0.153	7	18	52	1.3
3	55.0	0.126	....	0.126	0 195	9	22	60	1.5
4	74.7	0.164	....	0.164	0.254	9	22	60	1.5
5	102.0	0.220	....	0 220	0.341	10	23	60	1.7
6	136.0	0.285	....	0.285	0.442	9	22	60	1.5
7	263.0	0.526	0.25	0.776	1.20	8	20	55	1.6
8	302.2	0 59	....	0.84	1.30	9	22	60	1.8
9	347.0	0.68	....	0.93	1.44	8	21	57	1.9
10	374.0	0.71	....	0.96	1.49	9	22	60	2.0
11	561.0	1.04	0.37	1.66	2.58	9	21	57	2.3
12	590.3	1.09	....	1.71	2.65	9	22	60	2.4
13	638.0	1.14	....	1.76	2.72	9	21	57	2.4
14	932.0	1.59	....	2.21	3.42	9	22	60	2.9
15	948.8	1.60	....	2.22	3.44	9	22	60	3.0
16	990.0	1.61	....	2.23	3.45	9	22	60	3.0

fall, the slopes required to maintain the flow result in adequate depth below the surface, and it is possible and desirable to carry the sewer through the manholes with only the fall due to the slope of the sewer.

Analysis for Minimum Flow Conditions.[1]—An experienced designer can usually estimate, on the basis of experience and judgment and of his knowledge of the basic conditions, whether the provision of adequate capacity to meet future conditions and maximum rates of flow will result

[1] See also, Chap. III, p. 141, *et seq.*

in objectionably low velocities at times of low flow and in the early years. Until a sufficient background of experience has been acquired, the designer should analyze the design for minimum flow conditions; and it is not always safe for an experienced designer to omit such analysis.

Table 80 shows the method of analysis as applied to the design in Table 79. In such an examination, the actual minimum rate of sewage flow should be applied, if it is known; if not, it should be estimated from the present population and the known (or estimated) night rate of water consumption, a reasonable dry-weather rate of ground-water flow, etc. For the present example, the following assumptions have been made:

1. Present density of population one-half that assumed for ultimate development.
2. Night rate of flow of domestic sewage, 50 gal. per day per person.
3. Dry-weather infiltration of ground water, 1,000 gal. per day per acre.

On this basis, a curve of sewage flow per acre for present conditions, similar to that in Fig. 72, can be prepared, from which the minimum rates of flow shown in the table are computed.

The ratio of the minimum flow to the capacity of the sewer when full shows the percentage of the capacity utilized by the minimum flow. The corresponding percentage of depth and of velocity may readily be obtained from a diagram similar to Fig. 32, page 128, but prepared for values of D and S approximating those appearing in the design. This diagram was used in this example, though it strictly applies only to a circular section with $D = 1.0$ and $S = 0.005$.

It should be remembered, in using the figures thus obtained, that there is some evidence that the value of n in pipe sewers increases as the depth of flow decreases, that is, that the depths of flow might actually be greater, and the velocities less than those computed.

Finally, it remains to consider whether the minimum velocities and depths will be so low as to make the operation and maintenance of the sewer costly or troublesome; and if the conditions seem likely to be objectionable, to re-examine the design with the object of increasing the grades and perhaps decreasing the sizes of the sewers, or otherwise obtaining larger minimum velocities. If this cannot be accomplished, it may be necessary to provide for special flushing or for cleaning the sewers frequently during the earlier years of their use.

Alternate Projects.—It may sometimes be found that two or more alternate locations are practicable for some of the sewers, in some cases involving locations across private land. Frequently, the relative desirability of such alternate projects can be determined by inspection, particularly when no change in size of sewer is probable; but in other cases it may be necessary to compute the complete design for each project, and make comparative estimates of cost before a decision can be

reached. Unless there is a material advantage in cost or other condition resulting from a location through private property, it is generally advisable to build sewers in the public ways.

In order that no possibility of improving the lay-out may be overlooked, all possible alternate plans must be given consideration.

CHAPTER VII

PRECIPITATION

Storm-water Runoff.—The sizes of combined sewers and storm-water drains are determined primarily by the rates of rainfall and the available slopes for the sewers. The study of precipitation and the runoff from areas of different shapes, slopes, and surface characteristics should never be neglected by the engineer interested in sewerage works. Until about 1910 there was a general tendency among engineers to rely on various formulas for runoff, but about that time the belief began to spread through municipal engineering offices and among consulting specialists that there was need of more complete and more accurate knowledge of rainfall and runoff, upon which to base the calculation of sewer sizes.

Rates of Precipitation.—Many precipitation records give only the total amount of rainfall day by day, or, at most, the total precipitation during each storm, together with the times of beginning and ending. Such records are of slight value in the study of storm-water runoff. It is the maximum rate of precipitation lasting for a sufficient time to produce maximum runoff conditions which is of importance. Rates of precipitation can only be obtained from the records of recording rain gages. The use of such gages has generally been limited to the larger cities, including the more important Weather Bureau stations and engineering offices where runoff problems have been studied in detail, and until recent years very little trustworthy information relating to maximum rates of precipitation has been available.

Uniform and Variable Rates of Precipitation.—For periods of time exceeding a few minutes nearly all intense rainfalls occur at rates varying from time to time; a heavy precipitation at a uniform rate throughout the whole time is unusual. Sometimes there will be a downpour of high intensity followed by a period of rainfall at gradually decreasing intensities; sometimes this will be reversed and the greatest downpour will occur at the end of the period; but more frequently, particularly for the longer durations, the "intense rainfall"[1] consists of alternating periods of precipitation at higher and lower rates. In general, the higher rates of rainfall are more likely to occur near the beginning than near the end of the period of intense rainfall.

Obviously, the only way in which records of rates of precipitation can be compiled and used, except for single storms, is in the form of averages.

[1] For definition of intense or excessive rainfall, see p. 262.

The maximum rate of precipitation for a term of 30 min. would, therefore, be understood to represent the average rate corresponding to the greatest total precipitation in a 30-min. period. Under this definition it makes no difference if the rate of precipitation falls to zero during a portion of the period as long as the total, and, therefore, the average rate for the entire period, is a maximum.

Setting and Exposure of Gages.—The correct setting of a rain gage is of great importance. The exposure should be such that no objects which might interfere with the collection, by causing wind currents or otherwise, are within 50 ft. of the gage, and the collector ring or opening of the gage should not be more than 30 in. above the surface of the ground, which is the standard setting for the regular Weather Bureau rain gage. This last condition is one which it is often difficult to meet. Elevated gages usually show a considerably less collection than those with standard setting. If it is impossible to locate a recording gage substantially at ground level, a standard rain gage of the ordinary type should be maintained in the vicinity of the recording gage, and the records of the latter should be adjusted to accord with those of the standard gage. The following paragraphs from Circular *E*, Instrument Division of the U. S. Weather Bureau, entitled "Measurement of Precipitation," are pertinent in this connection:

Exposure of Gages.—The exposure of gages is a very important matter. The wind is the most serious disturbing cause in collecting precipitation. In blowing against the gage the eddies of wind formed at its top and about the mouth carry away rain, and especially snow, so that too little is caught. Snow is often blown out of a deep gage after once lodging therein.

Rain gages in slightly different positions, if badly exposed, catch very different amounts of rainfall. Within a few yards of each other two gages may show a difference of 20 per cent in the rainfall in a heavy rain storm. The stronger the wind, the greater the difference is apt to be. In a high location, eddies of wind produced by walls of buildings divert rain that would otherwise fall in the gage. A gage near the edge of the roof, on the windward side of a building, shows less rainfall than one in the center of the roof. The vertical ascending current along the side of the wall extends slightly above the level of the roof, and part of the rain is carried away from the gage. In the center of a large, flat roof, at least 60 ft. square, the rainfall collected by a gage does not differ materially from that collected at the level of the ground. A gage on a plane with a tight board fence 3 ft. high around it at a distance of 3 ft. will collect 6 per cent more rain than if there were no fence. These differences are due entirely to wind currents.

Since the value of the precipitation records depends so greatly upon proper exposure, particular care should be taken in selecting a place for the location of the gage, and every precaution should be taken to protect it from molestation. If possible, a position should be chosen in some open lot, unobstructed by large trees or buildings. Low bushes and fences, or walls that break the force of the wind in the vicinity of the gage, are bene-

ficial, if at a distance not less than the height of the object. The gages should be exposed upon the roof of a building only when a better exposure is not available; and, when so located, the middle portion of a flat, unobstructed roof enclosed by parapet walls generally gives the best results.

The Absolute Measurement of Rainfall.—It is generally conceded that the true catch of rainfall is obtained by the so-called pit gage; that is, a sunken collector, with its mouth elevated above the ground only far enough to prevent insplashing to any serious extent and set in the middle of a large open level field. Such a gage, however, easily becomes fouled with leaves and litter, and consequently its use is objectionable except as a standard of reference in experimental investigations. A better disposition is secured by forming a shallow pit, a foot or so deep, with the earth thrown up in a circular rim 6 or 8 ft. in diameter. The collector is placed at the center of the depression with its mouth about level with, or a little below the rim of the pit. Such a gage is so effectually sheltered from the wind that it collects the same quantity of rain as falls upon an equal area of the ground near by.

Nipher demonstrated in 1878 that almost or quite the true catch of rainfall could be collected in ordinary rain gages by surrounding them with a trumpet-shaped shield of sheet metal terminated in an annular rim of copper-wire gauze, 20 gage, mesh 8 wires per inch, to prevent insplashing. This device so far minimized the ill effects of the wind, that one of these shielded gages on a pole 18 ft. above the tower of the university and 118 ft. above the ground, collected the same amount of rainfall as a shielded gage on the ground. Hellmann and others have also found the Nipher screen useful, and have secured equally satisfactory results by the use of a fence or wind break around the top of the gage, at a distance from it equal to the height of the gage, and at an angular elevation above the gage of about 20 to 30 deg. These devices deflect and check the force of the wind at the mouth of the gage to such an extent that the raindrops can enter the gage in a normal manner, and a true catch be obtained.

It seems appropriate at this point to say that, while the Weather Bureau is compelled to expose rain gages upon the roofs of lofty buildings in large cities, the catch of rainfall thus obtained is often quite satisfactory. This is accomplished by taking advantage of the sheltering and protecting influences afforded by large parapet walls, which are generally to be found around flat-topped office buildings. Shields and guards upon the gages themselves in these cases are not so effectual, since the distribution of the rain over the roof is quite irregular. The whole building may be regarded as a huge, lofty rain gage. If shields and fences could be put around the building itself, a true catch might be secured, but in the absence of these, a gage located in the middle of the roof, especially if it is surrounded by parapet walls 3 or 4 ft. high, collects nearly the true amount of rain. Roof exposures are accepted by the Weather Bureau as an unavoidable necessity at its stations in the centers of large cities where better exposures are impossible. Ground exposures obtain wherever conditions permit, as, for example, in the smaller cities and at stations of co-operative and special rainfall observers.

From what has been stated it appears that the pit gage is probably the ultimate standard for the collection of rainfall and that a nearly true catch may also be obtained by the use of properly shielded gages.

Measurement of Precipitation.—Robert E. Horton[1] gives an excellent discussion of methods and gages used at various times and places, to which reference is made for description of standard rain gages with comments on their accuracy. This paper relates mainly to "ordinary" rainfall records, in which the total quantity of precipitation is the information sought. The description and discussion of recording gages is brief and incomplete. Types of recording rain gages are described hereinafter, with illustrations of the more important ones used in this country.

RECORDING RAIN GAGES

Requisites.—The principal requisites of a satisfactory recording rain gage are that it shall accurately record the quantity of rain failing and the time in which it falls. The measurement of the quantity may be obtained by gaging either the volume or weight of water, and its accuracy is dependent upon the correctness and adjustment of the measuring chamber, scales, or other apparatus, and their continuous and satisfactory operation. The character of the record is dependent upon the mechanism operating the pen or pencil, the clockwork, and the scales adopted for the record sheet. S. P. Fergusson[2] of the U. S. Weather Bureau says:

> The scales of the record sheet of a measuring instrument should be capable of indicating (1) the smallest value it is customary to measure, (2) the maximum range of the phenomenon to be measured, (3) the maximum rate for the smallest time interval usually found useful, and (4) the inclination or steepness of the line traced by the pen should not greatly exceed 45 deg., particularly if rates for small time intervals are to be measured. In the United States, the smallest quantity of precipitation usually measured with reasonable certainty is 0.01 in., the total in any storm rarely exceeds 6 in., and 5 min. is the smallest time interval ordinarily employed in the measurement of excessive rate . . . Lack of consideration of these points is more evident in the design of intermittent rain gages than in any other form, for there are numerous instances of instruments otherwise well designed where the scale of depth is 5 to 1 or even 10 to 1, while the time scale is not more than 10 mm., and sometimes not more than 2 mm., to 1 hour.

Types of Gages.—The principal types of recording rain gages are the following:

The *tipping bucket gage*, in which a bucket of two compartments is supported on trunnions, in such a way that the bucket will tip and thus empty one compartment as soon as it is filled, at the same time bringing the other compartment under the funnel. The number of tips or oscillations of the bucket is recorded on a chart, usually by electrical means. The Friez gage is a commonly used gage of this type.

[1] "The Measurement of Rainfall and Snow," *Jour. New Eng. Water Works Assoc.*, 1919; **33**, 14.

[2] "Improved Gages for Precipitation," *Monthly Weather Review* (July, 1921).

The *weighing gage*, which contains a receiver or can in which the rain is collected, and which is carried on scales or a spring balance so arranged that the quantity in the can is recorded upon a moving chart. The Fergusson gage is a good example of this type.

The *float gage*, which requires the use of a receiver of uniform horizontal section, in which the depth of water represents the depth of rain collected; and a float in this chamber carries a pen recording upon a moving chart. The Marvin gage is a good example of this type.

The *pressure gage*, in which the quantity of water in the container is indicated by the use of a recording pressure gage made to show very small pressures. There are no commercial gages of this type, but one may be improvised without difficulty.

Descriptions of Particular Gages.—*The Friez gage* is made by Julien P. Friez and Son, Baltimore, Md. In this instrument (Fig. 73) rain is collected in a funnel 12 in. or 10.5 in. in diameter and conducted through a tube into a bucket containing two compartments. The contents of each compartment are equivalent to 0.01 in. of rain. The bucket is supported on trunnions in such a manner that as soon as a compartment is filled it tips and discharges the accumulated rain, presenting the other compartment for filling. Each time the bucket tips it makes an electrical contact and causes a pen to record a step upon a chart carried by a revolving cylinder (Fig. 74). A sample chart from this instrument is shown in Fig. 75. It is seen that the curve traced does not represent directly the progress of the storm, the motion of the pen being reciprocating, up for 0.05 in. and down for 0.05 in.

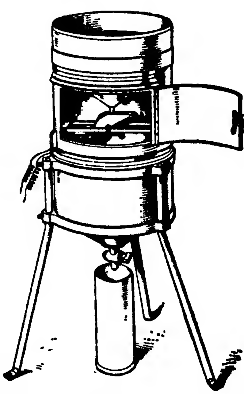


FIG. 73.—Friez tipping bucket gage.

The time scale of this chart is 2½ in. to an hour. The amount of rainfall is indicated, not by measurement on the chart, but by counting the number of steps, or of "flights" of 10 steps each. It is, therefore, possible to determine the rates of rainfall from this record with a very good degree of precision. The possibilities of error are, however, considerable. The Weather Bureau carefully investigated the accuracy of the instrument and determined that on account of the appreciable time required for the bucket to tip, the error due to inflow of water into a compartment already full before the bucket could tip and present the empty compartment is sufficient to produce an error of about 5 per cent at times of very heavy rain. It is found, also, that dirt washed into the buckets from the dust accumulating in the gage affects the character of the surfaces and the accuracy of the record. The adjustment of the

instrument must be carefully made and its record is absolutely dependent upon the electrical apparatus working correctly.

A test of such a gage, made by J. B. F. Breed, Chief Engineer of the Commissioners of Sewerage of Louisville, Ky., showed a total discrepancy

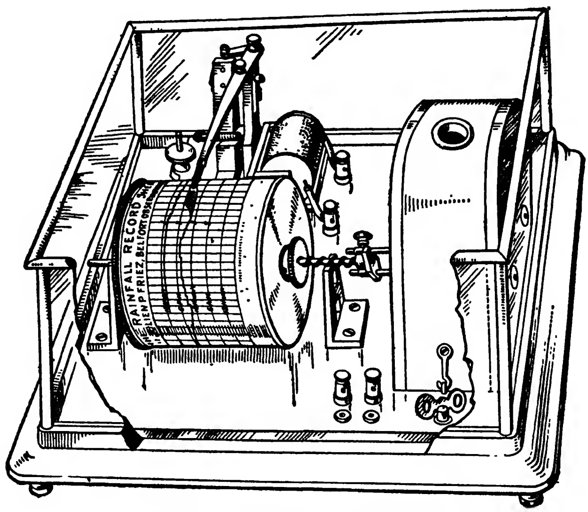
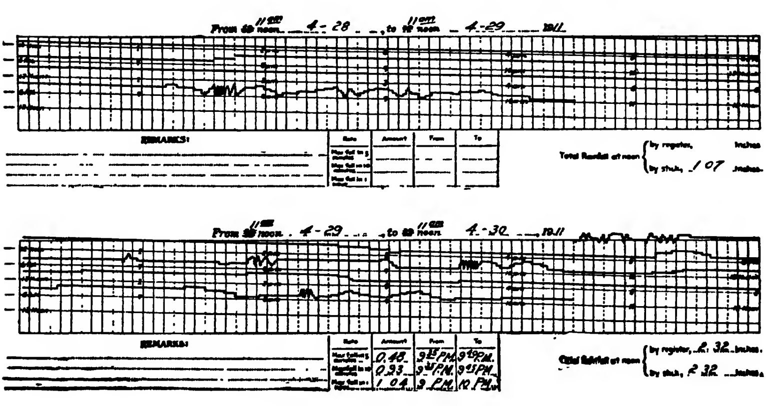


FIG. 74.—Register for Friez gage.



The chart is 10¼ inches long.
FIG. 75.—Chart from Friez gage.

of 17 per cent in a rainfall amounting to a total of 1.70 in. with a maximum intensity of 7.68 in. per hour for 5 min. and an average intensity of 1.65 in. for 60 min. By discharging water into the gage at various

rates and measuring the actual accumulation as compared with the recorded collection, it was found that the rates of precipitation computed from the gage record should be increased by about 2 per cent for each inch per hour. Thus a record showing precipitation at the rate of 5 in. per hour should be corrected by adding 10 per cent, making the corrected rate 5.50 in. per hour. The test was carried to an observed rate of 8.40 in. per hour, the actual rate being 9.78 in. per hour. It has also been

FIG. 76.—Fergusson rain gage.

found that the bucket sometimes stops on center, thus entirely failing to register, as a portion of the water flows out each side and the bucket no longer tips.

The Fergusson gage (1921 pattern) is also made by Friez. It is shown in Fig. 76, and a copy of the chart in Fig. 77. The water received by the collector is discharged into a can supported upon a spring balance, the movement of which is transmitted by link motion to a pen moving through an arc of a circle and making a record upon a chart carried by a revolving drum. This drum is made large enough to provide a satisfactory time scale and, as the levers are arranged so that the pen moves back and forth across the sheet until it has crossed it four times, it is

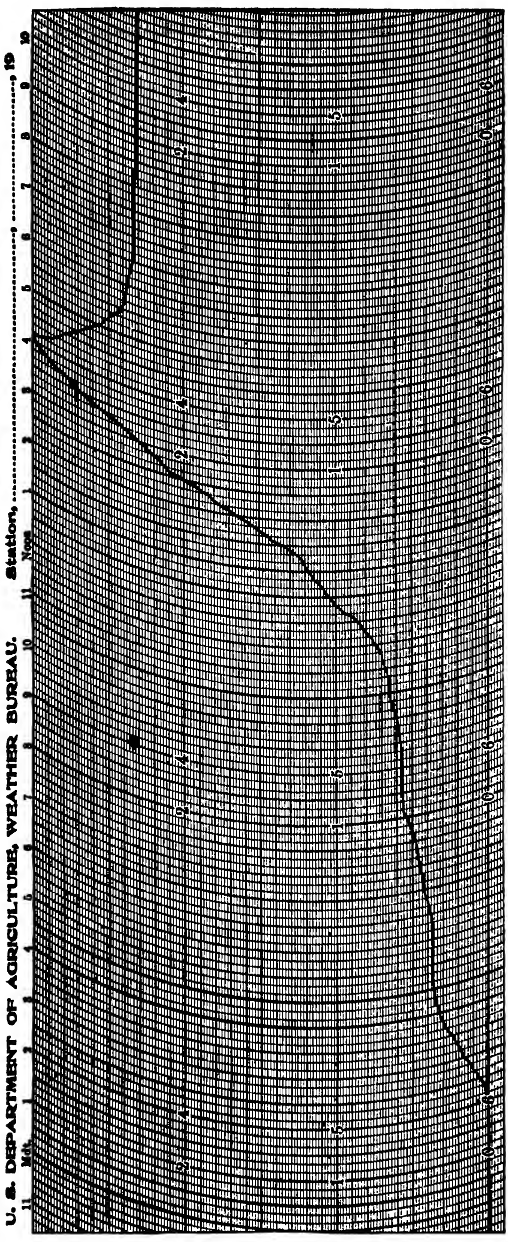


FIG. 77.—Chart from Fergusson rain gage.

possible to show the precipitation on a magnified scale (1½-in. on chart to 1 in. rain) and to record a total of 6 in. of rain.

The old (1888) pattern of the Fergusson gage has been widely used. It is similar to the present gage, and has a capacity of 6 in. of rain which is shown on natural scale, while the time scale is but ½ in. per hour. The mechanical details of the old gage were not nearly as good as those of the new one.

In spite of its disadvantages, some valuable rainfall intensity records have been obtained by the use of the old pattern *Draper gage*, in which

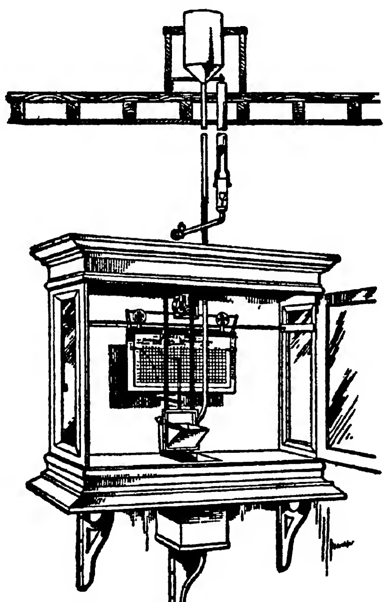


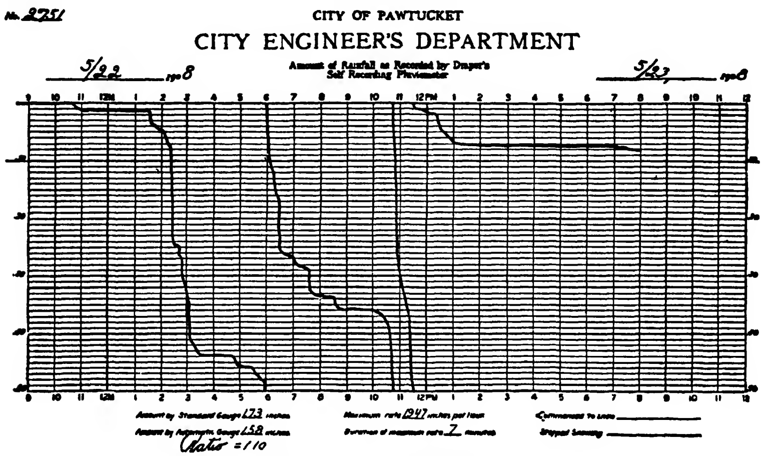
FIG. 78.—Draper rain gage, old style.

the water was collected through a funnel placed in the roof of the building or chamber and conveyed through a tube into a tipping bucket suspended from two helicoidal springs (Fig. 78). These springs were so adjusted that the scale of precipitation was magnified ten times, that is to say, 1 in. on the chart represented 0.1 in. of rain. The pen arm was attached to the bucket and moved directly with it, from the top of the chart to the bottom. When 0.5 in. of rain had been collected the bucket dumped and immediately returned to its upright position, bringing the pen to zero at the top of the chart.

The chart (Fig. 79) was carried on a flat plate suspended from a track and moved by clockwork. As originally constructed, the chart was

made to represent 1 week of time, but more recently the clockwork has been modified so that the chart makes a complete traverse in 24 hours. It is, however, so short, the total length being 12 in., that the scale is very small, only ½ in. per hour.

By this instrument the precipitation is unnecessarily magnified, while the time scale, as in the case of most instruments on the market, is too small for accurate determination of high rates of rainfall. The necessity of placing the collector upon the roof of the building, or of constructing a vault for the recorder, is also objectionable.



The chart is 12 inches long.

FIG. 79.—Chart from old-style Draper gage.

An improvement of the old type of Draper gage, devised by George A. Carpenter,[1] city engineer of Pawtucket, R. I., makes possible the exact determination of 2-min. intervals of time, and, accordingly, the maximum rates of precipitation for very short periods can be accurately determined. This result is accomplished by tapping the pen at intervals of 2 min. in such a way as to make a dash crossing the line traced by the pen and therefore dividing the chart record into sections each 2 min. long. Thus the amount of precipitation in each 2 min. can be read from the chart with great accuracy.

The *Marvin float gage* was designed by Charles F. Marvin, Chief of the U. S. Weather Bureau, and is made by Friez. It is somewhat elaborate and comparatively costly, but has certain advantages, the most important of which are that it enables the record of 8 days to be made on a single sheet, while avoiding the errors of the tipping-bucket type of gage and being independent of electrical circuits for its operation. The

[1] *Eng. News*, 1915; **74**, 148.

pen moves back and forth over a width of ½ in. on the chart, changing direction for each ¼ in. of rain.

Clock Movements.—The importance of a good clock movement in a recording rain gage is not always recognized. It is, however, a point that should receive careful consideration in the selection of an instrument. Not only should the clock be carefully regulated to keep correct time, but it is of much importance in any work involving two or more automatic gages, that the clocks be accurately synchronized, as otherwise it is impossible to draw any trustworthy conclusions relating to travel of storms, or to the time interval between rainfall and runoff in sewers. It is very desirable that all clocks be provided with dials to facilitate regulating and synchronizing, and in important work of large extent the practicability of electrically controlled clocks should be considered.

TIME-INTENSITY RELATION

It is well known that, in a general way, the intensity of precipitation varies inversely with the duration of the downpour, or in other words, that very heavy showers do not last as long as rains of less intensity. This variation in intensity, however, was not widely recognized as significant until recording rain gages had been used to a considerable extent. Until recent years, no considerable amount of trustworthy information on intensity of precipitation was available, since practically all rainfall records included little more than the total precipitation in each storm, or at most the time of beginning and ending of the storm in addition to the depth of rain. Moreover, not until the establishment of automatic or recording rain gages became somewhat general, and until these had been maintained for a sufficient period to get records of some length, was there sufficient information on which to predicate definite statements as to the relation between the intensity of rainfall and the length of the period during which the rain might fall continuously at any given rate. Little such information was available before the beginning of the present century.

Relation between Intensity and Duration of Rain.—One of the earliest attempts to determine the relation between the intensity and duration of precipitation was that of Prof. F. E. Nipher of St. Louis, who, studying the records for that city for a period of 47 years, and analyzing such information as he could find relating to the heavier storms, reached the conclusion that this relation could be shown by the formula $i = 360/t$.[1]

In 1889, Emil Kuichling, investigating the rainfall in the vicinity of Rochester, N. Y., similarly studied such records as were available, and reached the conclusion that in Rochester, for rains lasting less than 1 hour the intensity might be expressed by the formula $i = 3.73 -$

[1] *Am. Eng.*, 1885.

$0.0506t$, and for periods longer than 1 hour and less than 5 hours, the intensity would be $i = 0.99 - 0.002t$. To Kuichling's studies is due, in large measure, the present development of the rational method of estimating storm-water runoff.

In 1891, Prof. A. N. Talbot analyzed in detail the rainfall records reported by the United States Weather Bureau and some from other sources. The greater part of them were records of ordinary rain gages, but in a few cases the records were those of recording gages maintained in the larger cities. From this study he concluded that, for that part of the United States lying east of the Rocky Mountains, the formula $i = 360/(t + 30)$ would express the maximum rainfall which was ever likely to occur, and the formula $i = 105/(t + 15)$ would indicate the intensity of as heavy rains as it would ordinarily be necessary to consider in engineering design. Talbot's studies show very few storms indeed giving intensities higher than those showed by the first formula, but a considerable number higher than the second equation.

From 1900 to 1915 or thereabouts, a considerable number of curves were developed for various localities in the United States. These were usually derived by plotting as ordinates several of the highest observed rates of precipitation for a given time interval, against the corresponding time as abscissa, and repeating this for various time periods from 5 to 120 min. or more. The time-intensity curve was sometimes drawn as an envelope, including all the highest points, in which case it represented the maximum intensities of precipitation observed at the station under consideration; or it might be drawn somewhat below, and generally parallel to, the enveloping curve, representing in the judgment of the engineer the intensity of precipitation for which it was reasonable to make provision in designing drains. Such curves have generally been suggested as "bases of design," without attempting to indicate the extent of the inadequacy which might accompany their adoption. In some cases, three curves were derived, assumed to represent the maximum, rare, and frequent rainfall conditions.

Form of Rainfall Curve.—In many cases where mathematical expressions have been obtained for the rainfall curve, they have been written in the form, $i = a/(t + b)$. This formula has the advantage of being easily solvable by simple arithmetical operations; and if the constants in it have been correctly determined, it usually expresses the actual observations with a fair degree of accuracy between the limits of 10 min. and 2 hours' duration of rain. For either longer or shorter periods of time, however, the results obtained from this form of curve are generally too low. Practically, this is usually of little moment, since it is rarely necessary to consider a shorter period of time than 10 min. or a longer one than 2 hours in the design of sewers. It is, however, desirable, if the curve is to be expressed in mathematical form, to have this form as

nearly correct as possible, and the exponential form, $i = a/t^b$ has been found to fit the recorded observations with a good degree of accuracy in many cases. The exponent b is usually found to be between 0.5 and 0.7. When it is just 0.5, this equation can be solved as easily as the former one, since the results can be obtained directly with a single setting of the slide rule for each value of b.

The form $i = \dfrac{a}{\sqrt{t + b}}$ is also a convenient one for use, as it can be solved nearly as easily as the equation just mentioned, and, at least in some cases, it fits the observed points more satisfactorily. It has the further advantage that the value of i as it approaches zero is a finite quantity, its magnitude depending on the values chosen for a and b.

FREQUENCY OF RAINFALL AT HIGH RATES

It is usually financially impracticable, and sometimes physically impossible, to provide drains large enough to carry away, without delay, the runoff from the heaviest rainfall which may ever occur. It is, therefore, of great importance to have at least an approximate idea of the relative frequency of rains of various intensities. The economical period for which the design should be made is discussed on page 174.

Development of Time-intensity-frequency Curves.—If records of intense rainfalls are available for a sufficient period, say 20 years, so that they may be assumed to include all the weather conditions which are ordinarily experienced, then the maximum observed intensity for any duration of rain may be taken as that likely to be equalled or exceeded once in that period, in this case once in 20 years; the second has been equalled or exceeded twice in the period of record, or on the average once in 10 years, and may be assumed to have a frequency of 10 years. Similarly, intensities likely to occur in any time interval shorter than the period of record may be obtained.

If then the highest 10 points for each time interval (duration of downpour) be plotted, the lines connecting the several points of the same rank will represent the intensities which have been *equalled or exceeded* on the average once in 20 years, 10 years, 6⅔ years, 5 years, and other intervals, down to once in 2 years. For convenience, the corresponding rainfall rates are said to have frequencies of 20 or 10 years, and the like. Judging the future by the past, and smoothing out the curves so as to eliminate obvious inconsistencies (as for instance when a point is so far in excess of others as to indicate an occurrence unlikely to recur in a much longer period), the resulting curves may reasonably be taken as time-intensity curves of various degrees of frequency.

Obviously, the lower curves are less likely to be in error than the outermost or enveloping curve; and the longer the period of record, the better

the series of curves. Some of the maximum points in a record, say of 30 years, for example, may diverge so much from the others as to indicate that they really belong on a curve of much less frequency, such as a 50- or 100-year curve. On the other hand, the highest two or three curves may be so close together as to indicate that the highest is probably not the true average curve for the stated period of years.

In an analysis of this kind, there is no way of estimating the probable frequency of such abnormal data. It is customary in such cases to omit from consideration any points which are so far above the others as to indicate that they really represent the maximum for a much longer period of time than that covered by the record. The "probability method" outlined on page 269 is a better method for utilizing such data.

Direct plotting of the data for each duration of downpour may be employed as a test of the reasonableness of some of the assumptions. Logarithmic coordinate paper is particularly advantageous for such plotting. Such a record as that in Table 82, for instance, would be tested by plotting a smooth curve for 5-min. duration of downpour, in which ordinates would represent intensity of precipitation and abscissas the average frequency of occurrence; another curve for 10-min. duration, and so on. It will be seen that the maximum observed intensities are considerably below the curve corresponding to the other points—so much so as to indicate that these maxima probably correspond more nearly to 15- than to 24-year average frequency. It would be possible from the smooth curves on this diagram to construct hypothetical curves of time-intensity-frequency relations; and for the highest intensities occurring during the period of record, this would be more likely to be correct than the method usually followed.

If the curves for the various durations are in reasonable agreement with the plotted points, it may be possible to extend them and to construct a curve of probable relations corresponding to a frequency much less than the term of the observations; for instance, a 50-year curve might be based upon records covering 20 years. The dangers of relying too much upon such extrapolated curves are obvious.

The application of this method may be illustrated by an example abstracted from a Report to the Commissioners of Sewerage of Louisville, Kentucky, by J. B. F. Breed and Metcalf and Eddy, dated Aug. 31, 1921.

Intensity of Rainfall at Louisville, Kentucky.—The United States Weather Bureau has maintained a rain gage in Louisville since 1872. Since 1894 a registering automatic rain gage has been in use. The records from 1894 to 1897 inclusive for excessive rainfalls[1] are recorded in

[1] *Excessive rainfalls*, as defined by the U. S. Weather Bureau, include all rates of precipitation in excess of $(0.01t + 0.20)$, *i.e.*, time in minutes (taken as hundredths of an inch) plus 0.20. Thus 0.25 in. in 5 min., 0.80 in. in 60 min., and 1.40 in. in 120 min., are excessive rainfalls.

TABLE 81.—INTENSITY, DURATION, AND FREQUENCY OF RAINFALL, AT LOUISVILLE, KY.
Twenty-four years, 1897–1920

Number of times equalled or exceeded		1	2	3	4	5	6	7	8	9	10
Average frequency, years		24	12	8	6	$4\frac{4}{5}$	4	$3\frac{3}{7}$	3	$2\frac{2}{3}$	$2\frac{4}{10}$
Duration, minutes											
5	Amount[1]...	0.77	0.74	0.61	0.57	0.49	0.48	0.45	0.42	0.41	0.38
	Intensity[2]...	9.24	8 88	7.32	6.84	5.88	5.76	5.40	5.04	4.92	4.56
10	Amount. .	1 15	1 14	0.87	0 76	0.69	0.67	0.66	0.65	0.64	0.61
	Intensity..	6.90	6 84	5 22	4.56	4.14	4.02	3.96	3.90	3.84	3.66
15	Amount..	1.40	1.33	1 05	1 00	0.95	0.90	0.84	0.78	0.77	0.77
	Intensity.	5 60	5.32	4.20	4.00	3.80	3.60	3.36	3.12	3.08	3.08
20	Amount..	1 59	1 44	1 24	1.16	1.14	1 09	0 96	0.96	0.93	0.90
	Intensity. .	4.77	4 32	3 72	3.48	3 42	3.27	2.88	2.88	2.79	2.70
25	Amount. .	1.68	1 49	1 45	1 33	1 32	1.14	1.10	1.05	1.03	1.00
	Intensity..	4.03	3 58	3 48	3 19	3.17	2.74	2.64	2.52	2.47	2.40
30	Amount..	1.72	1 71	1 54	1 44	1 44	1.24	1.13	1.10	1.09	1.09
	Intensity..	3 44	3 42	3.08	2.88	2.88	2.48	2.26	2.20	2.18	2.18
45	Amount..	1.82	1 81	1 69	....	1.47	1.45	1.39	1.29	1.27	1.24
	Intensity..	2.43	2.41	2 26	....	1.96	1.93	1.85	1.72	1.69	1.65
60	Amount.	2.00	1.93	....	1.73	1 53	1.47	1.41	1.39	1.36	1.33
	Intensity...	2.00	1 93	....	1.73	1.53	1.47	1.41	1.39	1.36	1.33
80	Amount...	2.43	2 38	1.99	....	1.58	1.57	1.57	1.57	1.53	1.49
	Intensity...	1.82	1.79	1.49	....	1.19	1.18	1.18	1.18	1.15	1.12
100	Amount..	2.70	2.44	2.02	....	1.67	1.65	1.61	1.61	1.60	1.58
	Intensity..	1.62	1.46	1.21	....	1.00	0.99	0.97	0.97	0.96	0.95
120	Amount...	2.72	2.46	2.05	....	1.73	1.72	1.72	1.66	....	1.62
	Intensity..	1.36	1.23	1.03	....	0.87	0.86	0.86	0.83	....	0.81
180	Amount.	3.01	2.49	2.28	2.13	2.09	1.86	1.85	1.85	1.80	1.74
	Intensity..	1.00	0.83	0.76	0.71	0.70	0.62	0.62	0.62	0.60	0.58
240	Amount...	3 16	....	2 48	2 25	2.14	2.10	2.03	1.93	1.90	1.89
	Intensity..	0.79	....	0.62	0.56	0.54	0.52	0.51	0.48	0.48	0.47
300	Amount....	3.21	2.60	2.52	2.28	2.20	2.19	....	1.98		
	Intensity...	0.64	0.52	0.50	0.46	0.44	0.44	....	0.40		
360	Amount....	....	2.82	2.69	2.38						
	Intensity. .	....	0.47	0.45	0.40						

[1] Amount in inches.
[2] Intensity in inches per hour.

the Monthly Weather Review, the maximum amount being given for the 5-, 10-, and 60-min. intervals. Since 1897, records of accumulated amounts have been published for several time intervals, after the beginning of intense rates. In this study, the 24 years' records for the period 1897–1920 are used.

The maximum amounts and intensities are given in Table 81, arranged in the order of their magnitude and number of occurrences for the periods indicated, regardless of the storm in which they occurred; that is, the amounts and intensities shown for a given number of occurrences did not necessarily occur during a single storm. For example, the highest amount for 5-min. duration occurred during the storm of July 15, 1919, and for 10-, 15-, 20-, 25-, 30-, 45-, and 60-min. during the storm of Aug. 29, 1917, and for durations longer than 60 min. during the storm of Aug. 8, 1898. Therefore a sewer system, if designed to care for the runoff from rainfall intensities as shown for a given number of occurrences, would be likely to be surcharged only in part by a single storm in which intensities corresponding to those of a smaller number of occurrences were experienced.

The first line in the table shows the order or number of occurrences, and applies to the amounts and intensities given in the columns below the numbers. The figures represent the number of times that the intensities and amounts for the different periods have been equalled or exceeded in the 24 years covered by the record.

The second line shows the frequency in years and is equal to the length of the record divided by the number of occurrences.

In column 1 is the highest amount in inches, and the corresponding intensity in inches per hour for the duration stated. In column 2 is the second highest amount in inches and the corresponding intensity in inches per hour for the duration stated, and so on for the other columns.

It will be noted that no data are given for the storm of 360-min. duration and occurring once in the period of the record, although the storm occurring twice is shown. Similarly, for 45-min. duration, storms occurring three and five times are shown, but none for four occurrences. In these points, the table departs from the data obtained from the records, because the 360-min. precipitation tabulated as of two occurrences was actually experienced but once, and the other blanks indicate similar shifts of the record data. But if the cumulative amounts of rain be considered, it is obvious that the typical storm of 24-year frequency could not have yielded 3.21 in. in 300 min. and a smaller amount, 2.82 in., in 360 min. The quantity for 360 min. must be at least 3.21 in., unless the amount for 300 min. is abnormal and should be reduced, and the other figures give no indication that this is the case. The amount of 2.82 in., is, therefore, shifted to the column of amounts equalled or exceeded twice, subject to the same criterion when examining the data in that

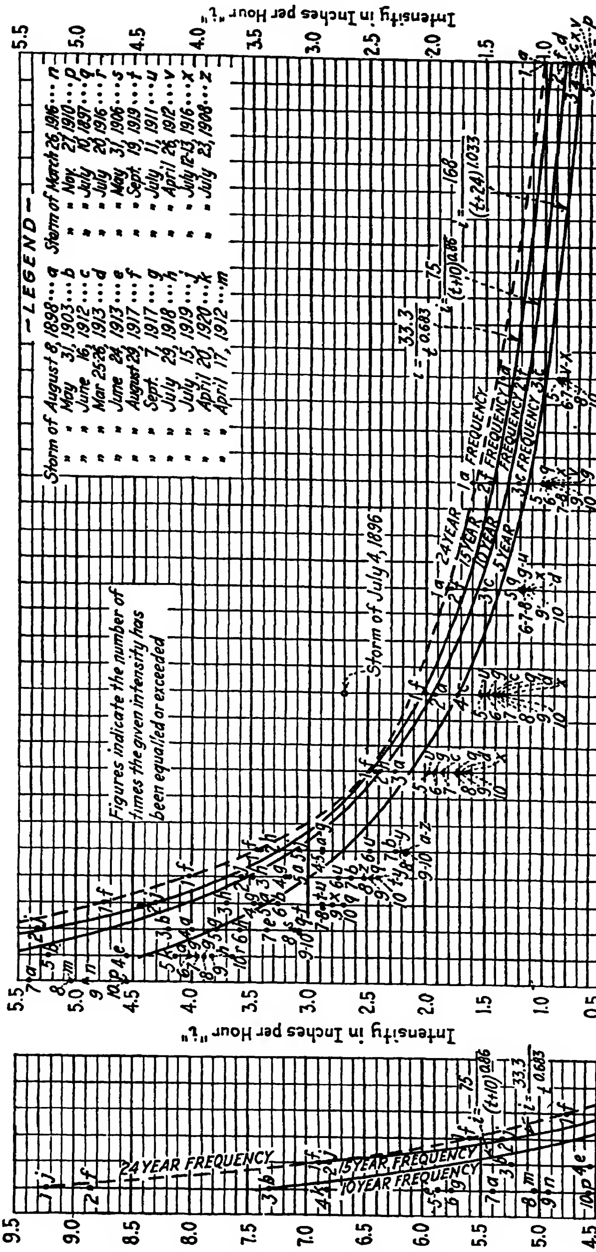


FIG. 80.—Intensity, duration and frequency of rainfall, Louisville, Ky., from U. S. Weather Bureau records 1897–1920 (24 years).

column. In a similar way, other figures have been shifted into columns later than their actual places, before utilizing the figures for drawing curves.

It sometimes happens that when two nearly contiguous downpours with moderate intensity of precipitation between them are treated as a single storm, somewhat higher intensities will be found for a given duration than for a shorter duration. This is very exceptional, however, and is not in accordance with the general relation which exists between intensity and duration of rainfall.

From the data in Table 81, Fig. 80 has been plotted with intensities in inches per hour as ordinates, and duration in minutes as abscissas. On this figure is also shown a point plotted for 2.70 in. per hour and for a duration of 60 min., and marked "Storm of July 4, 1896." As can be seen, this storm lies far above that of any in the records of 1897 to 1920, inclusive. It may, therefore, be inferred that this storm is of very unusual character and will recur only at very long intervals. It is quite certain that the city would not be justified in building all its sewers of a capacity sufficient to carry off the rainfall from a storm of such magnitude. The data for this storm are not as complete as for the other storms, as the maximum intensities for the 5-, 10-, and 60-min. periods only are given. Table 82 gives the data on this and other excessive rains reported in the *Monthly Weather Review* for 1895 and 1896.

Table 82.—Amounts and Rates of Excessive Rainfall at Louisville (Inches) 1895–1896

Date	Duration					
	5 min.		10 min.		60 min.	
	Amount	Intensity	Amount	Intensity	Amount	Intensity
June 4, 1895.....	0.50	6.00	0.55	3.30		
June 20, 1895....	....	....	....	....	0.87	0.87
June 23, 1896....	0.55	6.60	1.00	6.00	1.37[1]	1.37[2]
July 4, 1896.....	0.55	6.60	1.05	6.30	2.70	2.70
July 20–21, 1896.	....	....	. .	....	1.29	1.29

[1] Duration, 36 min.
[2] Rate for 36 min. = 3.80.

It is recognized that were the data for these storms complete, they might alter the arrangement in Table 81 and change the curves slightly. Because of the paucity of data, however, these storms have not been included in the analysis of the rainfall.

On Fig. 80 is shown the 24-year frequency curve, based upon the actual records. Curves for frequencies of 15, 10, and 5 years have also been deduced from the data and are shown on this figure.

The equation $i = 33.3/t^{0.683}$ gives results which correspond closely with intensities which may be expected to be equalled or exceeded on the average once in a period of 15 years. The 10-year curve is fairly well represented by the equation $i = 75/(t + 10)^{0.86}$. The 5-year curve is represented by the equation $i = 168/(t + 24)^{1.033}$.

It appears that there is very little difference between the 15- and the 10-year curves, especially for periods of 20 to 60 min., which are frequently used in the design of sewers and are, therefore, particularly significant. The variation is less than 10 per cent, so there would probably be no difference in the sizes adopted for sewers, whichever curve were used. It is of interest, however, to notice that within the significant period from 20 to 60 min., the 10-year curve is from 15 to 20 per cent higher than the 5-year curve. The record indicates that for periods of time greater than 30 min., sewers designed by the 5-year curve would be surcharged, on the average, three times, and if designed by the 10-year curve, twice in a period of 24 years.

The 15-year frequency curve is controlled by 5 storms. For durations of 5 to 25 min., inclusive, the storms of July 15, 1919, and Aug. 29, 1917, fix the location of the curve within narrow limits. For 30- and 45-min. duration the storms of July 29, 1918, and Aug. 29, 1917, determine the position of the curve. For durations from 60 min. to 180 min., inclusive, the storms of Aug. 8, 1898, and Aug. 29, 1917, determine the position of the curve. In general, the 15-year curve corresponds to the intensities experienced during the storm of Aug. 29, 1917. It lies somewhat below this storm for periods from 10 min. to 30 min., almost coincident with it for periods of 45 and 60 min., and is again a little below for durations of 80 min. and longer.

It will be noticed from Fig. 80 that the highest two intensities for a duration of 45 min. are almost coincident, and the 10-, 15-, and 24-year frequency curves are very close together. It seems probable that the city will experience an intensity for 45 min. somewhat higher than any shown by the records. This justifies the position of the 15-year curve which lies slightly above the maximum intensity which has been recorded as indicated on the figure for this duration.

Rainfall Intensities in Other Cities.—Table 83 shows the intensities for the St. Louis, Cincinnati, New York, and Boston 15-year curves, the Springfield 13-year curve and the Louisville 15-, 10-, and 5-year curves, and the Detroit 10-year curve.

A St. Louis curve, represented by the equation $i = 56/(t + 5)^{0.85}$, was derived by W. W. Horner, now engineer in charge of Division of Design, Sewers, and Paving, in 1910 from the U. S. Weather Bureau Records for

TABLE 83.—COMPARISON OF ST. LOUIS, CINCINNATI, NEW YORK, BOSTON, SPRINGFIELD, MASS., LOUISVILLE AND DETROIT RAINFALL INTENSITY—DURATION—FREQUENCY CURVES

Place	St. Louis	Cincinnati	New York	New York	Boston	Springfield, Mass.	Louisville			Detroit
Frequency, years	15 to 20	15	15[1]	15[2]	15[3]	13	15	10	5	10
Formula		$i = \frac{16}{t^{1/2}}$	$i = \frac{185}{t + 17}$		$i = \frac{20.4}{t^{0.61}}$	$i = \frac{250}{t + 38}$	$i = \frac{33.3}{t^{0.683}}$	$i = \frac{75}{(t + 10)^{0.86}}$	$i = \frac{168}{(t + 24)^{1.033}}$	$i = \frac{131}{t^{0.9} + 20.5}$
Length of record, yrs.	42	16	45	51	26 W. B.[4] 38 C. H.[5]	26	24	24	24	
Duration, minutes	Intensity, inches per hour									
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)
10	5.55	5.06	6.85	5.50	4.95	5.21	6.91	5.69	4.39	4.60
15	4.75	4.13	5.78	4.60	3.91	4.71	5.23	4.70	3.80	4.10
20	4.18	3.58	5.00	3.90	3.28	4.31	4.30	4.02	3.37	3.71
25	3.72	3.20	4.40	3.45	2.86	3.97	3.69	3.52	3.02	3.39
30	3.33	2.92	3.94	3.15	2.56	3.68	3.25	3.15	2.72	3.15
45	2.50	2.38	2.98	2.40	2.00	3.01	2.47	2.39	2.12	2.55
60	2.00	2.06	2.40	1.95	1.68	2.55	2.03	1.94	1.72	2.17
80	1.55	1.79	1.91	1.60	1.41	2.12	1.67	1.56	1.38	1.82
100	1.30	1.60	1.58	1.40	1.23	1.81	1.44	1.32	1.15	1.56
120	1.15	1.46	1.35	1.20	1.10	1.58	1.27	1.14	0.99	1.38
180	0.87	1.19	0.94	...	...	...	0.96	0.82	0.69	1.03
240	...	1.03	...	...	...	...	...	...	0.53	0.82
300	...	0.92	...		...	...	...	...	0.43	0.69

[1] Progress report of Committee on Rainfall and Runoff, Society of Municipal Engineers, New York City, 1913, Central Park record 1869 to 1913, inclusive.
[2] Eng. *News-Record*, 1921; **86**, 588.
[3] This curve is the same as the 5-year frequency curve for Springfield, Mass.
[4] Weather Bureau.
[5] Chestnut Hill.

the years 1873 to 1909, inclusive. This curve was used by Horner in sewer design in St. Louis from 1910 to 1915, and was estimated to represent intensities which would be equalled, or exceeded, on the average once in 15 years. In 1910, the city installed its own rain gages and the records of these gages, together with those originally obtained from the Weather Bureau, later made it advisable to revise the curve first adopted.

In 1915, another thorough study of the rainfall record was made, and the curve was again changed slightly and this curve (column 2, Table 83) was used in 1916 for all sewer design. On the basis of this curve it is estimated that if other considerations of the design are correct, the sewers will be surcharged somewhat at intervals of about 15 to 20 years.

The Cincinnati 15-year curve, represented by the equation $i = 16/t^{1/4}$ was derived from the U. S. Weather Bureau records from 1897 to 1912, and is given in the report of H. S. Morse and H. P. Eddy to H. M. Waite, Chief Engineer, Department of Public Works, dated Dec. 31, 1913.

The New York 15-year curve represented by the equation $i = \frac{185}{t + 17}$ (column 4) was derived from the 45-year record of the Central Park gage, 1869 to 1913, inclusive, and is given in a progress report of the Committee on Rainfall and Runoff of the Society of Municipal Engineers of New York City, published in the *Transactions* of the Society for 1913.

The New York 15-year curve given in column 5 is the curve proposed by Kenneth Allen, Sanitary Engineer, Board of Estimate and Apportionment, New York City, and is based on a probability study of the 51-year record of the Central Park gage,[1] 1869 to 1920.

The Boston[2] and Springfield[3] curves were derived by Metcalf and Eddy from the records of the Chestnut Hill and U. S. Weather Bureau gages at Boston and the municipal gage at Springfield.

Estimation of Time-intensity-frequency Relation on Basis of Probability.—Another method of estimating the time-intensity relation is the probability method, the application of which can be facilitated by the use of paper ruled in graduations proportionate to the relative probability of occurrence of events diverging from the mean by various amounts, according to the mathematical theory of probability—the so-called "probability paper." The application of this method in studying the frequency of excessive rains involves disregarding all except the greatest downpour in the unit time period adopted for the study; that is, if the period be taken as one year, the percentage of the years in which a certain intensity of rain may be expected can be estimated; but the possible occurrence of two or more excessive rains within a single year is left out

[1] Shown graphically in article published in *Eng. News-Record*, 1921; **86**, 588.
[2] *Jour. Boston Soc. C. E.*, 1920; **7**, 47.
[3] *Eng. News-Record*, 1920; **85**, 445.

of consideration. Only the maximum intensity in each year for each time of duration is used in the analysis; any secondary maxima in the same year are neglected. The method is therefore theoretically unsound, as some of the significant data are disregarded. It is claimed by advocates of this method that, practically, this is of little significance.

An application of this method to the Louisville rainfall record is shown in Table 84 and Fig. 81. The maximum rate of rainfall for each duration, which occurred in each year of the record, was selected

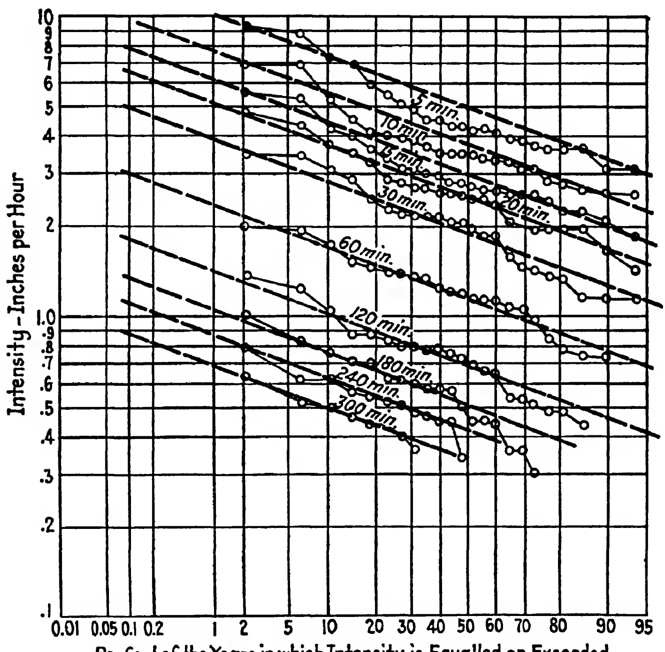


FIG. 81.—Excessive rainfall at Louisville, Ky. Probability study.

and tabulated in Table 84, and plotted in Fig. 81 on probability paper. Average lines drawn through the points thus plotted may be extended to indicate occurrences of much less frequency than the record itself would show.

In plotting such data, it is customary to assume that the figures for each year are representative of a strip of a width equivalent to the ratio of one year to the total number of years of the record. For a 24-year record, one year is 4.16 per cent; and if the maximum record is assumed to represent the average for 4.16 per cent of the years of a record of indefinite length, it should be plotted at 2.08 per cent on the diagram.

TABLE 84.—MAXIMUM INTENSITIES OF RAINFALL EXPERIENCED IN EACH OF THE YEARS 1897 to 1920, INCLUSIVE, AT LOUISVILLE, KY., ARRANGED IN ORDER OF THEIR MAGNITUDE

Occurrences		Average per cent for plotting	Intensities of precipitation equalled or exceded in minutes									
Times	Per cent		5	10	15	20	30	60	120	180	240	300
1	4.2	2.1	9 24	6.90	5.60	4.77	3.44	2.00	1.36	1.00	0.79	0.64
2	8.3	6.2	8.88	6.84	5.32	4.32	3.42	1.93	1.23	0.83	0.62	0.52
3	12.5	10.4	7.32	5.22	4.20	3.72	3.08	1.73	1.03	0.76	0.62	0.50
4	16.7	14.6	6.84	4.56	4.00	3.48	2.88	1.53	0.87	0.71	0.56	0.46
5	20 8	18.8	5.88	4.14	3.60	3.27	2.48	1.41	0.86	0.70	0.54	0.44
6	25.0	22 9	5.40	4.02	3.36	2.88	2.26	1.39	0.83	0.62	0.52	0.44
7	29.1	27.1	5.04	3.90	3.12	2.79	2.20	1.36	0.79	0.62	0.51	0.40
8	33 3	31.2	4.92	3.84	3.08	2.70	2.18	1.33	0.79	0.60	0.48	0.36
9	37.5	35.4	4.56	3.66	3.00	2.67	2.18	1.24	0.77	0.58	0.47	
10	41.6	39.6	4.56	3.54	2.96	2.58	2.16	1.24	0.77	0.58	0.45	
11	45 9	43 7	4.32	3.48	2.80	2.55	2.08	1.20	0.75	0.57	0.45	
12	50.0	48 0	4.32	3.48	2.76	2.55	2.06	1.18	0.73	0.50	0.34	
13	54 2	52 1	4.20	3.42	2.72	2.49	1.92	1.15	0.70	0.45		
14	58 3	56 3	4.20	3.36	2.64	2.49	1.86	1.12	0.66	0.45		
15	62 5	60 4	4.08	3.30	2.60	2.34	1.86	1.09	0.64	0.44		
16	66 7	64 6	3.96	3.24	2.56	2.07	1.60	1.08	0.54	0.36		
17	70.8	68 8	3.84	3.12	2.52	2.04	1.58	1.05	0.53	0.36		
18	75 0	72 9	3.72	3.12	2.52	1.96	1.46	0.97	0.51	0.30		
19	79 1	77 1	3.60	2.82	2.40	1.95	1.42	0.84	0.48			
20	83.3	81 2	3.60	2.76	2.24	1.95	1.36	0.77	0.48			
21	87.5	85 4	3.60	2.64	2.24	1.95	1.16	0.74	0.43			
22	91 7	89 6	3.12	2.58	2.08	1.65	1.14	0.73				
23	95.8	93 7	3.12	2.58	1.84	1.41	1.14	0.61				
24	100 0	97 9	No intense storm reported in one year of series									

Table 85 shows a comparison of 15-year time-intensity curves for Louisville as derived by the ordinary method of direct plotting, and by the probability method; and also a 100-year frequency time-intensity curve derived by the probability method.

It is seen that the probability study gives lower intensities for this curve than those obtained by direct plotting; but the figures probably ought not to be compared directly, since the probability study indicates rates which should be expected to recur in 1 year out of 15, or $6\frac{2}{3}$ per cent of the years—possibly two or more times in some of the years, and therefore more frequently than once in 15 years. Moreover, the comparison might be different for curves of other frequencies.

When occurrences of much greater rarity are considered, as once in 100 years, the possibility of such events occurring more than once in a single year is so slight that it may be left out of the reckoning. Nevertheless, the omission of secondary excessive rainfalls in years when two or more such storms occurred may affect the slope of the lines on the probability diagram. For extrapolation, as to estimate 100-year frequency from a 24-year record, the probability study may be the best method now available.

TABLE 85.—COMPARISON OF INTENSITY OF RAINFALL FOR VARIOUS DURATIONS AT LOUISVILLE TO BE EXPECTED ONCE IN 15 YEARS, AS ESTIMATED BY DIRECT PLOTTING AND BY PROBABILITY PAPER; ALSO INTENSITIES TO BE EXPECTED ONCE IN 100 YEARS, FROM PROBABILITY STUDY

Duration of downpour, minutes	Intensity to be expected once in 15 years		Intensity to be expected once in 100 years
	By direct plotting	By probability study	By probability study
	Inches per hour	Inches per hour	Inches per hour
10	6.91	6.0	7.7
15	5.23	4.7	6.1
20	4.30	4.0	5.1
30	3.25	3.0	3.9
60	2.03	1.85	2.35
120	1.27	1.10	1.40
180	0.96	0.82	1.05

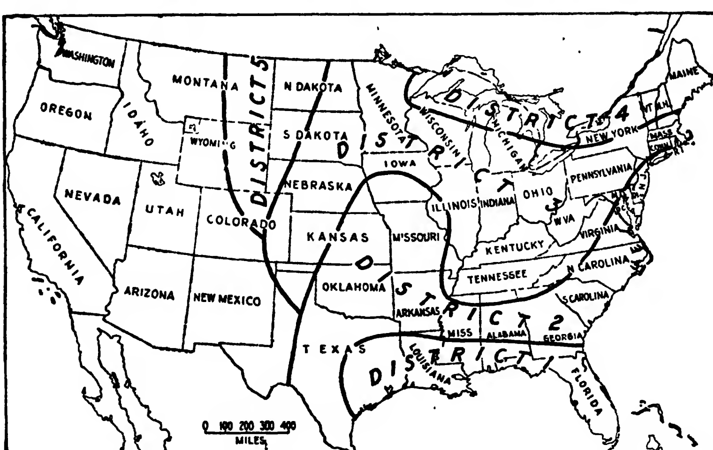


FIG. 82.—Districts for which Meyer's rainfall formulas were derived.

Time-intensity of rainfall curves for various parts of the United States were prepared for the U. S. Housing Commission.[1] Other curves may be found in reports similar to the Louisville report quoted above, and in articles in the technical press.

[1] John W. Alvord, Chief Engineer, *Monthly Weather Review*, August, 1921.

Meyer's Formulas.—In "The Elements of Hydrology," (1917) Prof. A. F. Meyer has developed formulas for intensity of rainfall in various sections ("groups") of the eastern half of the United States, utilizing the records of all the important rainfall stations in each group, as though they represented a single record of a length equal to the sums of the lengths of the individual records. He did not utilize any records for longer periods than 2 hours. From these data he has devised the formulas given in Table 86 as applicable for the districts shown on the map in Fig. 82.

TABLE 86.—MEYER'S FORMULAS FOR INTENSITY OF RAINFALL IN VARIOUS SECTIONS OF THE UNITED STATES, FOR THE STATED FREQUENCIES
See map (Fig. 82) for limits of sections

District	Intensity of rainfall, in inches per hour, for frequency of						
	1 year	2 years	5 years	10 years	25 years	50 years	100 years
1. Bordering Gulf of Mexico..	$\frac{145}{t + 23}$	$\frac{180}{t + 24.5}$	$\frac{220}{t + 27}$	$\frac{276}{t + 32}$	$\frac{355}{t + 40}$	$\frac{450}{t + 50}$	$\frac{600}{t + 65}$
2. Central Texas, Eastern Kansas, Missouri, Arkansas, Northern Mississippi, Georgia, and Atlantic Coast to Narragansett Bay..........	$\frac{100}{t + 18}$	$\frac{131}{t + 21}$	$\frac{171}{t + 23.5}$	$\frac{214}{t + 26}$	$\frac{252}{t + 28}$	$\frac{289}{t + 30}$	$\frac{325}{t + 32}$
3. East of Meridian 100 and north of District 2 as far as Great Lakes, Southern New York, and Southern New England..................	$\frac{72}{t + 13}$	$\frac{96}{t + 16}$	$\frac{122}{t + 18}$	$\frac{150}{t + 19.5}$	$\frac{181}{t + 21}$	$\frac{216}{t + 23}$	$\frac{256}{t + 25}$
4. Canada East of Meridian 90, Northern New York, Northern New England	$\frac{60}{t + 15}$	$\frac{84}{t + 16}$	$\frac{108}{t + 17.5}$	$\frac{132}{t + 19}$	$\frac{160}{t + 20}$	$\frac{186}{t + 21}$	$\frac{210}{t + 22}$
5. Eastern Montana and Wyoming, Western Dakotas and Nebraska...........	$\frac{60}{t + 13}$	$\frac{75}{t + 13}$	$\frac{90}{t + 13}$	$\frac{105}{t + 13}$	$\frac{126}{t + 14}$	$\frac{152}{t + 16}$	$\frac{180}{t + 18}$

PHENOMENAL RAIN STORMS—"ACTS OF GOD"

Storms of extreme intensity, commonly called "cloudbursts," are occasionally experienced in the Eastern United States. Several very severe storms may even occur within a few weeks or months, at a given locality.

During 1913, New York City experienced four storms, in all of which the intensity of precipitation, practically throughout the storm, was greater than that given by the equation $i = 15/t^{0.5}$. The significant facts relative to these storms and intensities obtained by this formula are contained in Table 87.

Table 87.—Phenomenal Rainfalls in New York City, 1913

Date	July 10	July 29	Sept. 5	Oct. 1	
Place	100 Broadway	Central Park	Central Park	Richmond	$i = \frac{15}{\sqrt{t}}$
t minutes	Intensity i-inches per hour				
1	....	....	....	8.40	15.00
2	....	....	....	8.10	10.60
4	....	....	....	6.45	7.50
5	9.88	6.12	7.20	....	6.72
7	....	....	....	6.24	5.68
10	7.56	5.76	6 90	5.64	4.75
15	6.52	4.80	6 36	5.16	3.88
19	....	....	....	5.05	3.45
30	4.18	2 96	5.24	....	2.74
37	....	....	....	4.84	2.47
49	....	....	....	4.75	2.15
59	....	....	....	4 44	1.95
60	2.30	2.73	3.31	...	1.94
85	....	....	....	3.80	1.63
106	....	....	....	3.37	1.46
120	1.28	1.56	1.85	....	1.37
123	....	....	....	3 06	1.35

These storms showed intensities in several cases far beyond the 100-year curve[1] for New York given by Kenneth Allen[2] and the storm of Oct. 1 showed an intensity for 120 min. nearly 70 per cent higher than that given by Allen's "Absolute Maximum Curve." Allen's curves are given in Fig. 83.

Even more intense rainfalls are occasionally reported, but it seems probable that, outside the tropics, their occurrence in any given locality is extremely rare. Ivan E. Houk[3] quotes old records of a phenomenal rain of Aug. 5, 1843, which apparently included a downpour of about 16 in. in 3 hours at Concord, Delaware County, Pa. Robert E. Horton[4] reported such a rain at Taborton, N. Y., on Aug. 10, 1920, when 11.62 in. fell in 24 hours, of which 8.95 in. fell in about 2 hours.

Table 88 contains figures of the maximum intensity of precipitation for various durations during a number of heavy rains in various parts of the United States east of the Rocky Mountains. They do not neces-

[1] Based on a study of the 51-year record of the rain gage at Central Park.
[2] *Trans. Am. Soc. C. E.*, 1922; **85**, 121.
[3] *Eng. News-Record*, 1922; **89**, 402.
[4] *Monthly Weather Review*, April, 1921.

sarily represent the most severe storm of which there is record at each locality; indeed it is rarely the case that the greater intensities for all durations of downpour occur in a single storm.

The figures show that at least in the eastern half of the United States, rain storms of high intensity are of a similar character. They do not

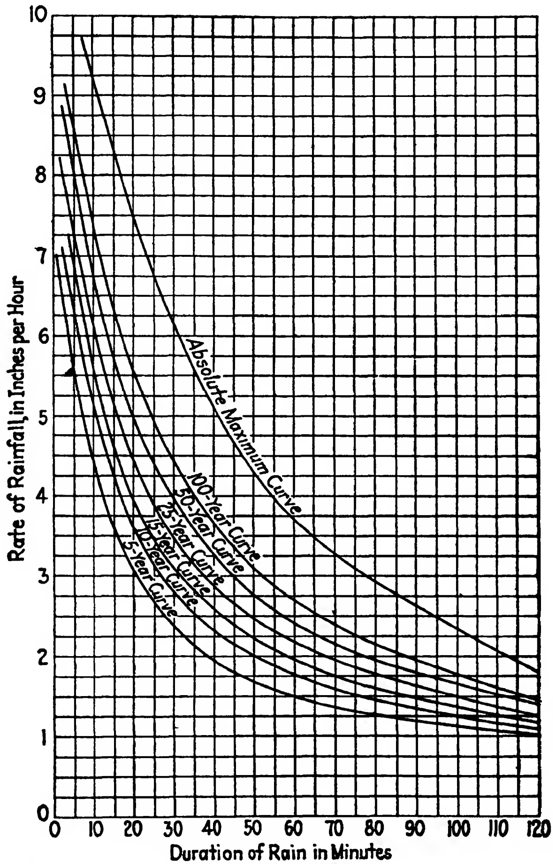


FIG. 83.—Maximum rainfall intensities, New York City, as determined from the records of the Central Park gage from 1869 to 1920. (*Allen.*)

give any indication of the relative frequency of such storms in various localities. In some places they may be experienced more frequently than once in ten years on an average; in other places they may be so rare as to be classed as "Acts of God."

A comprehensive definition of "Act of God" is found in the case of *U. S.* vs. *Kansas, etc., Ry. Co.*, 189 Fed. 471, 477, as follows:

An inevitable accident which could not have been foreseen and prevented by the exercise of that degree of diligence which reasonable men would exercise under like conditions and without any fault attributable to the party sought to be held responsible.

TABLE 88.—SOME PHENOMENAL RAINFALLS IN THE UNITED STATES
Intensity of precipitation in inches per hour

Duration, minutes	Boston Aug. 22, 1899	New York Aug. 12, 1926	Toledo Aug. 16, 1920	Detroit Aug. 24, 1926	Kansas City Aug. 23, 1903	Macon June 10, 1923	Pensacola Oct. 20, 1909	New Orleans April 15–16, 1927	Galveston Oct. 5–6, 1910
10	6.30	5.70	7.14	8.40	6.72	6.06	9.18		
15	5.77	5.56	7.04	7.44	6.60	5.76	9.16	6.56	
20	4.62	5.22	6.54	6.34	6.45	5.52	8.64	....	5.70
30	3.53	4.36	5.76	4.88	6.14	4.66	7.30	4.68	5.64
45	2.50	....	4.39	3.46	5.46	3.57	5.35	4.00	5.39
60	2.02	3.42	3.58	3.09	4.74	3.27	4.27	3.39	4.01
120	....	....	....	1.93	....	....	2.41	2.64	3.14
180	....	....	....	1.37	....	....	....	2.13	
240	....	....	....	1.11	....	....	....	1.84	
300	....	....	....	0.90	....	....	....	1.58	
480	....	....	....	....	....	....	....	1.09	

DISTRIBUTION OF INTENSE RAINFALL

The preceding discussion of intense precipitation and of time-intensity relations has been based upon the records of individual gages; that is, upon the rainfall at a definite point in each case considered. Where two or more gages have been placed a mile or more apart, it has been observed that, with intense storms, there is considerable difference in the rate and amount of rainfall recorded by adjacent gages. The area on which rain falls at a high rate for periods of an hour or less is comparatively small. In the drainage of large areas, there is possibility of considerable error in assuming the intensity of rainfall to be uniform and equal to that indicated by the time intensity curve for the locality under consideration.

Very little detailed information upon the distribution of intense rainfall is to be had. Few localities are even yet provided with an adequate number of recording rain gages so located as to show the travel of and area covered by storms. The eight recording gages in New York City are located on an area 29 miles long and 1½ miles wide; the three in Washington are in a line approximately parallel to the Potomac River; the seven in St. Louis are in an area about 6¾ miles long by 1¾ miles wide.

In the Boston Metropolitan District there are 13 recording rain gages within 10 miles of the State House, so located as to cover fairly well an

area of about 12 by 18 miles. Records of these gages during the years 1918 to 1922 have been compared and studied by Frank A. Marston,[1] who also made similar comparison of the records of one storm (in 1920) at New Orleans when all of the six gages maintained there were in operation, the data being furnished by George G. Earl, General Superintendent, Sewerage and Water Board. The records of a phenomenal storm at Cambridge, Ohio, in 1914, given in Meyer's "Hydrology," also were studied.

The data given indicate that, in a general way, the average intensity of precipitation, within the limits usually met in design, over an area of 1,000 acres is about 91 per cent of the maximum; and over an area of 5,000 acres, the average is about 83 per cent of the maximum. Approximate ratios of average intensity to maximum intensity for several periods of duration and various areas, as determined for Boston and New Orleans storms, are given in Table 89.

TABLE 89.—RATIOS OF AVERAGE INTENSITY OF PRECIPITATION OVER VARIOUS AREAS TO MAXIMUM WITHIN THE AREA, PERCENTAGE
Based upon storms at Boston, Mass., and New Orleans, La.

Area, acres	Duration, 15 min.	Duration, 30 min.	Duration, 45 min.	Duration, 60 min.
0	100	100	100	100
500	91	94	95	97
1,000	87	91	93	95
1,500	84	89	91	93
2,000	81	87	90	92
3,000	77	83	87	89
4,000	73	80	85	88
5,000	69	77	83	86

There is some justification for the belief that, irrespective of locality, downpours of equal duration and having the same maximum intensity of precipitation at the eye or focus of the storm will cover corresponding areas with about the same average intensity. If this proves to be true, data from storms occurring in New Orleans, or other places where precipitation of high intensity is of frequent occurrence, can be used to supplement data from other parts of the country, such as those from Boston, thereby adding greatly to the value of such records.

[1] *Trans. Am. Soc. C. E.*, 1924; **87**, 535.

CHAPTER VIII

STORM-WATER RUNOFF

Few problems have afforded the sewer designer more misgivings than the determination of the quantity of storm water for which storm drains or combined sewers should be provided. The chief reason for this lies in the fact that the problem is indeterminate, and that the information which may be available and the formulas which may be used only serve to aid his judgment, upon the soundness of which the correctness of the final solution largely depends. In fact, it is a difficult task to say when the solution of such a problem is correct within the usual meaning of the term, because no two engineers acting independently would be likely to reach the same conclusions as to the economic period in the future upon which to base the design of the system, the ultimate development and improvement of the district within this economic period, the rate of rainfall for which the community can reasonably be expected to provide drainage, and the rate at which the storm water will reach the sewers, all considerations vitally affecting the sizes of the drains or sewers being designed.

The earliest attempts to solve this problem were based upon observations or estimates of flow in existing streams, gutters, and drains. Formulas of an empirical character were derived from such studies, which are quoted and described in this chapter. Finally, the attention of engineers has been focused upon the fact that the runoff is directly dependent upon the rate of rainfall and the rapidity with which the water will reach the drains. This is a long step in advance; but the problem is still quite indeterminate and requires for its economic solution sound judgment aided by experience and by all the information which can be obtained.

Conditions Affecting Rate of Runoff.—The volume of storm water to be cared for by a sewer or drain depends upon the intensity and duration of the rain, and the character, slope, and area of the surface upon which it falls. Of the water falling upon the surface, a portion is lost by evaporation; still another is required to fill the depressions of the surface; another portion sinks into the earth, where it is either retained by capillary attraction or percolates slowly through the earth to reinforce the ground water and to reappear at some lower point in springs or streams; another portion is absorbed by vegetation; while the remainder

flows off over the surface until collected in natural or artificial channels. This last portion is the one with which the problem of storm drainage is concerned.

The proportion of the total rainfall which will flow off from any given area varies with the duration and intensity of the rain, with the degree of saturation of the earth before the storm, and with the condition of the surface of the ground, whether open or frozen or covered with snow or ice. It will also change from time to time for the same area as the character of the surface is artificially modified by the construction of streets, pavements, and buildings.

Assuming a rainfall of uniform intensity both as regards area and duration, it is evident that the runoff from any given district will be greatest when all parts of the district are contributing to the point under observation. To establish this condition requires a lapse of time, not only to allow the water flowing from the most distant part of the area to reach the outlet, but also to fill depressions and saturate the surface soil. For ordinary drainage districts which have relatively short periods of concentration and are not more than about 200 acres in extent, the assumption of uniform intensity and distribution of rainfall is substantially correct; therefore, the maximum runoff is to be expected from a rainfall of maximum uniform intensity lasting at least as long as the period of time required to allow the water from the farthest point of the drainage area to reach the outlet. For large districts with relatively long periods of concentration, the maximum flow during many storms occurs when some portions of the districts are contributing water at much smaller rates than other portions, because of the wide fluctuations in the intensity during the storm and upon different portions of the districts.

"RATIONAL METHOD" OF ESTIMATING STORM-WATER FLOW

At the present time the so-called rational method of estimating the quantity of runoff is commonly employed in the design of storm drains and combined sewers.

The rational method recognizes as axiomatic the direct relation between the rainfall and the runoff, as shown by the formula $Q = ciA$, in which c = a coefficient representing the ratio of runoff to rainfall, generally called the runoff coefficient; i = the intensity of rainfall in cubic feet per second per acre (or nearly enough, the rate of rainfall in inches per hour); A = the drainage area in acres.

In a computation by this method, the area A is definitely determined by measurement. It is also necessary to determine, first, the time of concentration, which is the length of time required to establish runoff

and for the water to flow from the most distant point of the district to the inlet, and thence through the drains to the point of observation; second, the greatest intensity of rainfall corresponding to this period of time, or, at least, the greatest intensity for which provision should be made in the design of the drains; and third, the runoff coefficient, which depends mainly upon the character of the soil, slope and character of the surface.

Time of Concentration.—As defined above, the time of concentration is the period of time required to establish runoff and for the water to flow from the most distant point (measured in time) to the point under consideration. It is made up of two parts, the inlet time and the time of flow in the drains. Inlet time is discussed in the following section. The time of flow in the drains is readily obtained by a simple hydraulic computation if the conditions, quantity of water, size, and slope of sewers, are known.

It is important to distinguish the *minimum* time of concentration from what may be called the *actual* time of concentration. The former corresponds to the condition for which sewers should be designed; conduits substantially full, velocity at a maximum, and conditions of surface such that runoff from roofs and streets and flow in gutters will be at maximum rates. Under these conditions, the time of concentration will be a minimum and the corresponding rate of precipitation will be a maximum. The conditions are, therefore, the most serious to which the drain may be subjected. The minimum time of concentration is a constant for a given sewer district in a particular state of development.

On the other hand, the *actual* time of concentration represents the time required for the concentration of the waters of a particular storm, under the conditions existing at the moment. If the storm is of moderate intensity, the drain may be but partly filled and the velocity of flow may, therefore, be considerably less than the maximum. Moreover, unless rain has previously been falling for some time, the filling of depressions and the accumulation of sufficient head to cause flow over rough or nearly flat surfaces will require an appreciable amount of time. The actual time of concentration will, therefore, exceed the minimum in all cases except those for which the drain was designed.

In problems of sewer design, the engineer is concerned only with the minimum time of concentration; but when gagings of storm-water flow are made, it is important to recognize that the conditions are reversed, and the flow must be compared with the precipitation causing it, which may have fallen in a time quite different from the computed time of concentration.

It is customary to estimate the time of concentration at any point by the cumulative addition of the inlet time and the computed time of flow in the several sections tributary at that point.

Time Required for Water to Reach the Drains (Inlet Time).—The time required to wet the surface, fill depressions and establish runoff and for the water to flow over the surface to the inlet and thence to the drain from those portions most distant from the inlet, is the inlet time commonly used in the design of drains by the so-called rational method. It must either be estimated from the available information or determined by observation. It will be larger for flat than for steep surface slopes, with deep building lots than with shallow ones and with irregular highly pervious surfaces than with smooth impervious surfaces. The spacing of inlets will also influence the inlet time. It will seldom be less than 3 or more than 20 min. In the case of small districts, or fairly large districts with steep slopes, this time is frequently the most important element in determining the quantity of water for which to provide. The inlet time as defined above should not be confused with the time of flow to the drain after the condition of runoff has been established. The inlet time ordinarily will be several minutes greater than the time of flow to the drain. The time of flow to the drain should be used in computing the actual time of concentration when applied to a period of rainfall following soon after a rainfall sufficient in amount and intensity to have established a condition of runoff. W. W. Horner states[1] that he has reached the conclusion, based upon observations, that the water from the streets, sidewalks and roofs will reach the sewer in from 2 to 5 min., with street grades of from ½ to 5 per cent (improved streets), but that the velocity over grass plots is very low and even in heavy rains from 10 to 20 min. will be required for the water to flow 100 ft.

Charles E. Gregory,[2] in his discussion of Grunsky's paper upon "The Sewer System of San Francisco," has computed theoretically the rate of runoff in a gutter 1,000 ft. long, having a slope of 0.0025, draining an impervious street surface 24 ft. wide, when there is a uniform rainfall at the rate of 4 in. per hour, and finds that this rate of precipitation would have to continue for 42 min. before the rate of discharge would equal the rate of precipitation, and that 25 min. would elapse before the rate of runoff equalled half the rate of precipitation. His conclusion is that for many roofs and a few street surfaces, where the storm-water inlets are moderately closely spaced, the common assumption of 5 min. as the time required for the storm water to reach the sewer at maximum rate may be true, but in most cases this time is materially greater and that it varies widely under different circumstances.

In view of the scarcity of definite information relating to individual sewer districts, the following information relating to the runoff from an area of 0.055 acre in a small city in Arkansas where the soil was heavy

[1] *Eng. News*, 1910; **64**, 326.
[2] *Trans. Am. Soc. C. E.*, 1909; **65**, 393.

and sun-baked, but without any paved or roof surfaces, is significant. This information was presented by James H. Fuertes.[1] He says:

Several years ago the opportunity was presented of measuring the run-off from a small tract of ground in a southern city. Although the observations were made with hastily improvised apparatus and the tract of ground was quite small, the writer offers it with suitable apologies for its meagerness, because of the scarcity of published records of such information for either large or small tracts. The tract of ground sloped quite uniformly in two directions toward one corner, the fall of the surface being about 5 ft. in 100 ft. Along one side a ditch was cut, into which the water drained from the whole area. At the end of the ditch a small weir was arranged, and the depth of the water flowing over the weir was measured with a thin ivory scale at as frequent intervals as the observations could be recorded, varying from 1 min. to about 3 min. The rain depths were similarly measured, though at less frequent intervals. The total depth of rain that fell upon the tract, in the particular storm in question, was 1.3 in. which fell in 37 min. The maximum rate of rainfall was 6 in. per hour, which continued about 5 min. and was reached 11 min. after the beginning of the storm.

At the beginning of the storm the ground was very hard and dry. The tract was a heavy, clayey soil, covered with a short and rather thin growth of grass. From the data obtained, it was deduced that 29 per cent of the total rainfall on the tract passed over the measuring weir; that the average velocity of the water in the ditch was about 4 ft. per second; and that the average velocity of the water flowing over the surface of the ground to the ditch was about 0.1 ft. per second.

The diagram accompanying this discussion shows that rain began at 6.40 and runoff at the gaging point at 6.47; maximum rainfall rate began at 6.51 and maximum rate of runoff was attained at 6.59; from which it may be deduced that the time of concentration for this area was at least 8 min.

The rain continued at the maximum rate of 6.0 in. per hour for but 5 min. The average rate of precipitation for the 8 min. of greatest rainfall was about 5.3 in. per hour, and the maximum run-off was 7.2 cu. ft. per minute, equivalent to 2.18 cu. ft. per second per acre, or 41 per cent of the rainfall rate for 8 min. The runoff factor was therefore 0.41.

Intensity of Rainfall.—The development of time-intensity-frequency curves of rainfall has been discussed in the preceding chapter. The considerations upon which to base the decision of the frequency curve to be used in design will be presented in the following chapter. Having adopted a rainfall curve, the rate of precipitation corresponding to the time of concentration is taken directly from the curve.

The rate or intensity of rainfall is used as though the precipitation occurred at a uniform rate throughout the period of concentration. As

[1] *Jour. West. Soc. Eng.*, 1899; **4**, 170.

a matter of fact, except for short periods of time, this is rarely the case. Sometimes an intense downpour is followed by a period of rain at gradually decreasing rates; sometimes the reverse is the case and the downpour comes at the end of the period; but more frequently the heavy rains of longer duration consist of alternate periods of intense and moderate precipitation.

If the rain falls at a uniform rate throughout the period, then, neglecting differences in runoff coefficient, the maximum runoff will occur at the end of the period, when rain which has just fallen on the area nearest the point of concentration is combining with that which fell upon the most distant points at the beginning of the period. The district may be considered as divided into zones from each of which the storm water will flow to the outlet in a uniform time, as 5, 10, or 15 min. If the precipitation occurs at varying rates which are uniform and of like duration over the entire area, then the outflow at the end of the period will be the result of the combination of water flowing at differing rates from the several zones, and if all the zones were of equal area (still neglecting differences of runoff coefficient between zones) the rate of runoff would still be the same as if the rain fell at a uniform rate over the entire area, equal to the average rate of precipitation for the time considered. Varation in the areas of zones, combined with variations in the intensity of rainfall, may result in a considerable difference in runoff from that resulting from a rain of uniform intensity throughout.

Runoff Coefficient.—The coefficient of runoff is difficult of exact determination, even for existing conditions, and is subject to great modification by artificial alterations in the condition of the surface, such as changes in the degree of development of the built-up district and in the proportion of the area covered by paved streets. It is, therefore, necessary in designing sewers to estimate the conditions which are likely to obtain a reasonable time in the future, unless the district under consideration has already reached such a degree of development that no further changes are probable.

The runoff coefficient depends upon a large number of elements and is not constant for a given area, even during a single storm. It was formerly considered that this factor represented strictly the actual percentage of impervious surface in the district under consideration, and that if the entire surface were covered with impervious materials, such as roofs and asphalt pavements, the factor would be 1.00. It is now recognized, however, that the factor is seldom unity, even for an absolutely impervious surface. Some evaporation always takes place, even during the progress of a rain storm, and small quantities of water are required for wetting impervious surfaces. Irregularities of the surface also tend to hold back some of the water and prevent its running off as rapidly as it falls.

In this connection, it may aid the engineer in forming a conception of the problem of runoff to consider the quantity of water actually falling in several periods of time, as given in Table 90 computed from the curve, $i = 15/t^{0.5}$.

TABLE 90.—QUANTITY OF RAIN FALLING IN THE SPECIFIED PERIODS OF TIME AT THE RATES INDICATED BY CURVE OF INTENSITIES, $i = 15/t^{0.5}$

Time, minutes	Rate of precipitation, inches per hour	Accumulated depth of precipitation, inches
5	6.71	0.56
10	4.75	0.79
15	3.88	0.97
20	3.36	1.12
30	2.75	1.38
45	2.24	1.68
60	1.94	1.94
90	1.58	2.37
120	1.37	2.74

Note that these periods of time are not necessarily measured from the beginning of a storm, or even from the beginning of the downpour. Prof. A. J. Henry[1] of the U. S. Weather Bureau gives a table showing the percentage of cases of downpour in Washington, Savannah, and St. Louis, in which the maximum rate of precipitation occurred at various periods after the beginning of the storm. This information is given in Table 91.

The runoff factor gradually increases for some time after the beginning of a rain until the soil has been thoroughly saturated and until impervious surfaces have been thoroughly wetted and the depressions filled. After that time the coefficient remains substantially constant for a given area. It therefore makes considerable difference in the amount of runoff whether the critical precipitation comes near the beginning of a storm or after rain has been falling for some time.

It is also possible, as previously stated, that if an excessive rain comes at a time when there is snow or ice upon the ground, the coefficient may be greater than unity, although this condition is so unlikely as applied to sewer design that it may ordinarily be left out of consideration.

The coefficient of runoff is really the product of four factors, which may be called the coefficients of imperviousness, of distribution of rainfall, of retention, and of retardation.

[1] "Rainfall of the United States," *Bull. D*, U. S. Weather Bureau; also, *Jour. West. Soc Eng.*, 1899; 4, 165.

In practice, it is seldom necessary to estimate these factors separately, except the coefficient of imperviousness; but it is important for the engineer to bear in mind the composite nature of the coefficient of runoff, and to give due consideration to the conditions affecting its several parts.

TABLE 91.—PERCENTAGE OF CASES IN WHICH THE MAXIMUM INTENSITY OF PRECIPITATION OCCURRED WITHIN VARIOUS PERIODS FROM THE BEGINNING OF THE STORM

Minutes after beginning of storm	Per cent of cases in which maximum intensity occurred within period at		
	Washington	Savannah	St. Louis
5	17	10	31
10	38	31	61
15	59	52	69
20	64	65	74
25	72	72	76
30	81	82	78
35	86	87	80
40	91	88	88
45	93	92	93
50	94	97	98
60	100	100	100

Coefficient of Imperviousness.—The determination of the proportion of the tributary area which may be assumed to be impervious is based, in part, upon an estimate of the proportion of the surface which will be covered by pavements, roofs, and other impervious substances, and in part upon an estimate of the relative imperviousness of the soil which is not so covered. Later these two estimates are combined, thus forming one assumed average coefficient of imperviousness for the area.

As sewers are designed to adequately serve the district some years after they have been constructed, it is necessary to estimate the extent and character of the development which will take place in the meantime. It is evident, therefore, that the determination of net imperviousness is largely dependent upon judgment aided by actual measurement of areas of roofs and pavements in districts which may be considered typical of the district under consideration at the end of the period for which the proposed sewer is to be designed. Data from a few such measurements are given in Table 92.

TABLE 92.—PERCENTAGE OF AREAS OCCUPIED BY RELATIVELY IMPERVIOUS SURFACES IN PARTS OF SEVERAL AMERICAN CITIES

City	Character of district	Size of block	Extent of development	Percentage of total area occupied by				
				Streets	Alleys	Private walks	Roofs	Total of impervious areas
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
Louisville, Ky.	Residential	470 by 470 ft.	100 per cent	24	4	2	12	42
Louisville, Ky.	Residential	235 by 410 ft.	100 per cent	31	...	....	16	47
Louisville, Ky.	Residential	455 by 650 ft.	100 per cent	28	4	....	21	53
Louisville, Ky.	Residential	480 by 480 ft.	67 per cent	23	...	7	10	40
Louisville, Ky.	Residential	480 by 480 ft.	90 per cent	22	3	5	21	51
Louisville, Ky.	Residential	480 by 480 ft.	67 per cent	23	8	2	10	43
Louisville, Ky.	Residential	480 by 480 ft.	95 per cent	19	8	6	23	56
Louisville, Ky.	Residential	470 by 555 ft.	100 per cent	18	5	6	23	52
Louisville, Ky.	Residential	475 by 845 ft.	100 per cent	17	5	6	24	52
Louisville, Ky.	Residential	535 by 970 ft.	100 per cent	18.5	3.5	1.5	8.7	32
Louisville, Ky.	Residential	430 by 560 ft.	100 per cent	23.2	4 6	1.9	16.5	46
Louisville, Ky.	Residential	520 by 800 ft.	100 per cent	18.8	3.5	1.2	9.1	33
Louisville, Ky.	Residential	380 by 450 ft.	95 per cent	26.6	4.5	2.7	18.4	52
Louisville, Ky.	Residential	600 by 800 ft.	100 per cent	17.9	3.3	1.9	11.9	35
Springfield, Mass.	Residential	256,000 sq. ft.	100 per cent	21.1	...	4.6	23.4	49
Cincinnati, Ohio	Residential	291.1 acres	20 persons per acre	15.6	...	3.2	16.2	35
Cincinnati, Ohio	Tenement and industrial	35.6 acres	55 persons per acre	23.5	...	3.8	28.0	55
Cincinnati, Ohio	Tenement	60.0 acres	135 persons per acre	28.0	...	12.2	43.9	84
Cincinnati, Ohio	Commercial	30.4 acres	100 per cent	32.2	...	1.3	66.5	100
Detroit, Mich.	Residential	7.2 acres	100 per cent	18	6	5	21	50
Detroit, Mich.	Residential	7.0 acres	100 per cent	17	5	5	15	43
Detroit, Mich.	Residential	6.5 acres	100 per cent	18	6	4	12	40
Detroit, Mich. (Grove Point Park)	Residential	9.4 acres	100 per cent	13	...	6	9	28
Quincy, Mass.	Residential	3.6 acres	100 per cent	14	...	2	16.5	32.5
Quincy, Mass.	Residential	4.6 acres	100 per cent	15	...	2	14	31
Quincy, Mass.	Residential	4.0 acres	100 per cent	18	...	3	21	42
Schenectady, N. Y.	Residential	7.4 acres	100 per cent	19	...	2	20	41
Schenectday, N. Y.	Residential	5.2 acres	100 per cent	17	...	2	16	35

Coefficient of Distribution of Rainfall.—Data upon the distribution of rainfall over a drainage area are few and inconclusive. Marston's studies, referred to in the preceding chapter, indicate that for the smaller drainage areas this coefficient is very nearly unity. The indications are that for areas of 1,000 acres this factor may be about 0.95; for 2,500 acres, 0.90; and for 5,000 acres, 0.85, for durations of about 60 min.

Coefficient of Retention.—This coefficient takes account of the water required to wet the surfaces; evaporation during a storm; water held back in depressions and irregularities of the surface, and by vegetation, etc., and water absorbed by porous earth, which, therefore, does not find its way into the sewers. All of these influences have vastly more effect at the beginning of a storm than after rain has been falling for some time, and also vary with climatic conditions, so that the value of this coefficient is far from constant, even for a single drainage area. Furthermore, in growing cities the extent of the areas covered by roofs and impervious pavements is continually increasing, with a corresponding diminution of more or less pervious areas, and pavements and roofs are being made smoother and less absorbent. For this reason, present values of the coefficient are of service only for comparative purposes.

Coefficient of Retardation.—If the duration of the storm causing flood conditions is less than the time required for water to flow from the most distant point on the drainage area to the point for which computations or gagings are made, then the maximum discharge will come when less than the whole drainage area is contributing water. The ratio of the area so contributing to the total drainage area is called the coefficient of retardation.

Obviously, if the precipitation continues at a uniform rate for an indefinite time, the greatest discharge will occur when all parts of the drainage area are contributing water, and at an interval after beginning of the downpour equal to the time required for water to flow from the most distant point (measured in time of flow) to the point under consideration. If the downpour lasts but a short time, and particularly if the drainage area is irregular in shape, it is possible that the maximum discharge may occur when but a portion of the area is contributing water. This portion will be the largest area within the drainage area and between two "contours" (lines of equal "time-distance" or equal time of flow from the point under consideration) whose distance apart, measured in time, is equal to the duration of the downpour. If this time should equal the time of concentration for the entire area, the ratio would be unity and there would be no retardation.

In problems of design, it is believed that (except, perhaps, in the case of very large drainage areas) the maximum discharge would result from a rain lasting for a sufficient period so that the entire area would contribute water—in other words, for a period equal to the time of con-

centration—and accordingly retardation should not be considered in such work. Since i varies inversely as $t^{1/2}$ (and roughly as $A^{1/4}$ for rectangular tracts), Q is a maximum when A is a maximum. This would hold true for coefficients of distribution of rainfall near unity. For large areas, this coefficient decreases till the relationship no longer holds and maximum flow may occur with only a part of the area contributing runoff from excessive intensities of rainfall lasting for periods less than the time of concentration for the entire area.

While American engineers have generally neglected retardation in design, it should not be lost sight of in studying gagings of flow in sewers. In other words, unless it is certain that the downpour has lasted for a period equal to or exceeding the time of concentration, it must be remembered that all parts of the drainage area may not have been contributing water to the maximum discharge, and the area which was actually contributing must be determined in order to find the true runoff factor.

In making this allowance for retardation, the effect of the travel of the storm should not be lost sight of. Information on this point is usually not to be had, but would be required for a complete and accurate solution of the problem.

It must not be forgotten that the time of concentration for a given drainage area is not a constant and will be greater in light storms, when the sewers are but partly filled, than in heavy storms, when maximum velocities are attained. This condition, like the "coefficient of retardation," is of major importance only in studying gagings and comparing them with the storms producing the runoff, since in sewer design allowance must be made for maximum conditions.

Effect of Storage in Sewers and upon Streets and Other Surfaces.—Still another element of retardation is found in the necessity of filling the sewer, gutters, and others channels to a sufficient depth and also to accumulate sufficient head to carry away the water entering the drains. Thus it will be seen that a certain portion of the precipitation which is really running off is temporarily stored or retarded and the rate of flow in the sewers is somewhat less than it would be if all the water could be conveyed away as rapidly as it is received.

Grunsky[1] has discussed this problem at length, and, making certain assumptions, has elaborated the rational method of designing storm-water drains with allowance for the storage capacity of the conduits themselves. His analysis indicates that, for any given period of concentration, something like one-half the total quantity of water which will ultimately run off is temporarily stored or in transit at the end of the period. As a rule, it is better in designing sewers to take no account of this storage capacity, leaving it as an additional factor of safety. The

[1] *Trans. Am. Soc. C. E.*, 1909; **65**, 294.

effect of such storage must, however, be borne in mind when studying gagings of storm-water flow and comparing them with precipitation records, although when a condition of uniform flow has been established, which will be the case when water is flowing off as fast as it enters the drains (at the end of the period of concentration), storage is not being changed and comparison of runoff and precipitation can be made directly.

The Swedish engineer, W. von Greyerz, has suggested that in some cases it may be economical to limit the size and reduce the number of storm-water inlets, thus forcing the water to flow in gutters for long distances and causing the gutters to serve as retarding basins. Obviously, the time of concentration would be materially lengthened if such a policy were adopted, and the size of drains required would be somewhat reduced. In general, ponding in gutters is not looked upon with favor, but under certain circumstances it may be justifiable or even advantageous. This is more likely to be the case in flat outlying districts than elsewhere. On steep slopes the possible pondage would be small, and the effect of keeping the storm water in the gutters would be felt in surface flooding at the foot of the slope.

Values Ordinarily Assumed for Runoff Factor.—In computing from observations of rainfall and runoff, the runoff coefficients for the various kinds of surface that are found in a given urban territory, much care must be exercised in the selection of the data. Ordinarily, the rainfall is neither uniform in intensity nor uniformly distributed over the district; in some cases, the discharge is estimated from inadequate data, and in others the areas of the several classes of surface are not determined with much accuracy. As a rule, also, the time of concentration corresponding to the conditions existing at the time of gaging has not been determined, so the intensity of precipitation with which the runoff should be compared is not known. It is, therefore, not surprising to find wide differences in the results obtained by different observers with respect to the coefficients of the several classes of surface. The range of such variation reported in textbooks and papers on the subject is exhibited in Table 93.

TABLE 93.—RANGE IN ESTIMATES OF RUNOFF FROM DIFFERENT CLASSES OF SURFACE IN PROPORTION TO THE RAINFALL INTENSITY

From Bryant and Kuichling's Report on the Adequacy of the Present Sewerage System of the Back Bay District of Boston, etc., 1909

For watertight roof surfaces	0.70 to 0.95
For asphalt pavements in good order	0.85 to 0.90
For stone, brick, and wooden block pavements with tightly cemented joints	0.75 to 0.85
For same with open or uncemented joints	0.50 to 0.70
For inferior block pavements with open joints	0.40 to 0.50
For macadamized roadways	0.25 to 0.60
For gravel roadways and walks	0.15 to 0.30
For unpaved surfaces, railroad yards, and vacant lots	0.10 to 0.30
For parks, gardens, lawns, and meadows, depending on surface slope and character of subsoil	0.05 to 0.25

Other authorities do not attempt to make close estimates for the different kinds of surface in an urban district, but content themselves with average values of the proportional runoff, as follows:

For the most densely built-up portions of the district	0.70 to 0.90
For the adjoining well built-up portions	0.50 to 0.70
For the residential portions with detached houses	0.25 to 0.50
For the suburban portions, with few buildings	0 10 to 0.25

According to Frühling, the values of this coefficient (assuming the surface already wetted by a previous rain) are about as follows:

For metal, glazed tile, and slate roofs	0 95
For ordinary tile and roofing papers	0.90
For asphalt and other smooth and dense pavements	0.85–0.90
For closely jointed wood or stone-block pavements	0.80–0.85
For block pavements with wide joints	0.50–0.70
For cobblestone pavements	0.4 –0.5
For macadam roadways	0.25–0.45
For gravel roadways	0.15–0.30

and for large areas there may be assumed:

For the densely built center of the city	0.7 –0.9
For densely built residence districts	0.5 –0.7
For residence districts, not densely built	0.25–0.5
For parks and open spaces	0.1 –0.3
For lawns, gardens, meadows, and cultivated areas, varying with slope and character of soil	0.05–0.25
For wooded areas	0.01–0.20

Dr. Karl Imhoff[1] gives for ordinary German conditions the "general assumptions relating to quantity of sewage" reproduced in Table 94.

[1] "Taschenbuch der Stadtentwässerung," 1925.

TABLE 94.—QUANTITY OF SEWAGE AND RUNOFF IN GERMAN CITIES

Class	Conditions	Population, per acre	Quantity of house sewage,[1] cubic feet per second per acre	Storm water	
				Coefficient of runoff	Rate of runoff,[2] cubic feet per second per acre
I	Very thickly built up..	141	0.0116	0.80	2.27
II	Closely built up.....	101	0.0083	0.60	1.70
III	Well built up......	61	0.0050	0.25	0.71
IV	Suburban..........	40	0 0033	0.15	0.43
V	Unsettled.........	0	0.0	0.05	0.14

[1] Based upon 100 liters per head per day, flowing off in 12 hours.

[2] Based upon a precipitation of 200 liters per second per hectare, equivalent to 2.84 in. per hour; for 15-min. duration and 10-year frequency.

Variation in Coefficient of Runoff with Duration of Rain.—The increase in runoff coefficient with the duration of the rainfall has long been recognized. Emil Kuichling reached such a conclusion in his paper on "The Relation between the Rainfall and the Discharge of Sewers in Populous Districts."[1] Charles E. Gregory in "Rainfall and Runoff in Storm-water Sewers"[2] derived a formula from his studies of some measurements by Hering of the runoff from a certain district in New York City and from published records of discharge measurements for storm-water sewers, for the relation between the runoff coefficient and the duration of the rainfall for impervious surfaces. This formula was expressed by $c = 0.175t^{1/3}$ where c is the runoff coefficient and t is the duration in minutes. The corresponding values of c for totally impervious surfaces for various times would then be

t............	3	5	10	15	20	30	45	60	90	120	180	186
c............	0.25	0.30	0.38	0.43	0.48	0.55	0.62	0.68	0.79	0.86	0.99	1.00

W. W. Horner[3] suggested values for the runoff coefficients for different durations for both impervious and pervious surfaces based upon gagings of the flow from small areas in St. Louis.

H. G. McGee[4] published a general formula by W. C. Hoad for the coefficient of runoff, $c = \frac{at}{b+t}$.

[1] *Trans. Am. Soc. C. E.*, 1889; **20**, 1.

[2] *Trans. Am. Soc. C. E.*, 1907; **58**, 458.

[3] *Eng. News*, 1910; **64**, 326.

[4] *Eng. News-Record*, 1919; **83**, 868.

Three specific formulas also are given as follows:

For impervious surfaces . . . . $c = \dfrac{t}{8 + t}$

For improved pervious surfaces . . . . $c = \dfrac{0.5t}{15 + t}$

For sandy, very pervious surfaces . . . . $c = \dfrac{0.3t}{20 + t}$

The above runoff coefficients are shown graphically in Fig. 84, together with some coefficients used by the authors in designing storm-water drains. For the "zone principle" employed in computing some of the coefficients, see the following section.

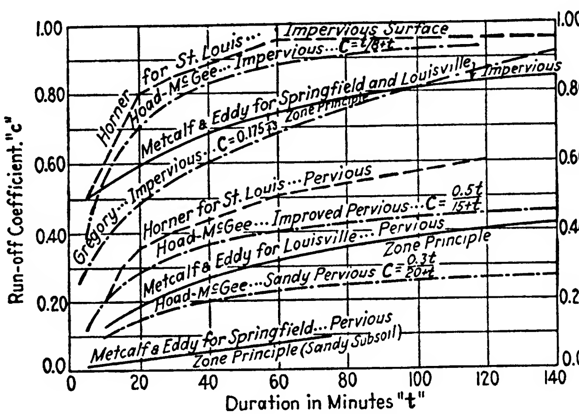


Fig. 84.—Runoff coefficients for various conditions and durations.

Basic coefficients of runoff must be chosen for the drainage area under consideration, either from the results of experience or by the use of a formula such as Gregory's, due allowance being made for the character of the subsoil and the degree of development. In the following table are shown runoff coefficients which were suggested by Horner for use in St. Louis, as a result of his experience and some measurements of flow to inlets made under his direction in that city.

The authors have used these or similar coefficients for several years, combined according to the "zone principle," and their experience indicates that the above values are reasonably close for impervious surfaces, but that those for pervious surfaces are applicable only where the soil is dense, and for sandy soils the coefficients for pervious surfaces should be smaller than those given above.

The basic coefficients are intended to apply to elementary areas where the time of concentration is relatively short, approximating a time commonly used as "inlet time;" thus, the runoff coefficient for a totally

Table 95.—Assumed Percentages of Runoff (Horner)

Duration t in minutes	Per cent runoff from	
	Impervious portion	Pervious portion
10	60	20
15	70	30
20	80	35
30	85	40
60	95	50
120	95	60

impervious area having a time of concentration of, say, 30 min., would not be 0.85, because at the end of 30 min. that portion of the flow which comes from the most distant point (30 min. distant) represents the first flow from a surface which had not been contributing flow up to that time. The total discharge is made up of the summation of the discharges from the various zones in which the coefficients decrease progressively from 0.85 to 0.50, as the distance from the outlet increases. The actual composite factor or average runoff coefficient for any drainage area will, therefore, depend upon the relative sizes of the zones and the relative proportion of impervious and pervious areas in each zone, and these conditions may vary widely.

Average Runoff Coefficient According to the "Zone Principle" (Time-contour Analysis).—If a drainage area be conceived as divided into zones by lines (called time contours), each connecting the points from which water will flow to the outlet in an equal time, then the water running off from any zone at any moment will be derived from a particular part of the storm, while the runoff from the next zone will be derived from a correspondingly earlier or later portion of the storm.

Obviously, the time contours are likely to be very irregular, being affected by irregularities of the surface, by surface slopes, by location of inlets, by slope and length of drains, and by other factors. For practical purposes, however, it is sufficient to consider the entire drainage area as approximating a regular geometrical figure—square, rectangle, triangle, or sector—and the zones as areas of equal width between arcs of circles having a center at the outlet. A drainage area having uniform velocities of flow over the surface and in the drains would have the lowest runoff coefficient, provided it approximated the shape of a sector of a circle with the outlet at the center; and a triangular area with the outlet at the shortest side would have the highest runoff coefficient, the reason being that, in the case of the sector, the greater proportion of the area is farthest from the outlet, while, in the case of the

triangle, the largest zone is nearest the outlet. A square with the outlet at the center of a side and a rectangle with the outlet in the center of the short side would have average coefficients intermediate between those of the sector and the triangle.

Figure 85 shows the relation between percentage of total area contributing to the flow at the outlet, and the percentage of the elapsed time

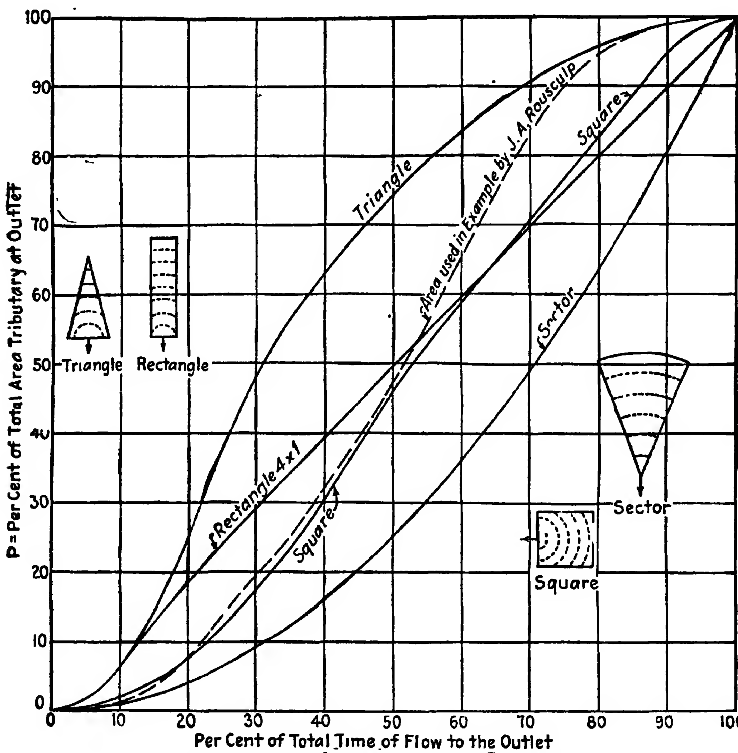


FIG. 85.—Ratio of area tributary at outlet to time of flow for regular areas with constant velocity.

(of the total time required for the entire area to contribute to the flow), for equal areas of the shapes and proportions shown, on the assumption of uniform velocities of flow from all portions to the outlet.

Areas with varying velocities of flow, whatever their actual shape, may resemble in runoff coefficients any of the above geometric figures, depending upon the relative times of flow from the different portions to the outlet. An inspection of the areas will usually indicate the effective

TABLE 96.—AVERAGE RUNOFF COEFFICIENTS FOR IMPERVIOUS SURFACES, RECTANGULAR AREAS IN WHICH THE LENGTH IS FOUR TIMES THE BREADTH. COMPUTED FROM BASIC COEFFICIENTS IN TABLE 95, COMBINED ACCORDING TO THE ZONE PRINCIPLE

In this table P represents the percentage of the total tributary area

Duration of rain on zone in min.	Run-off coefficient for one zone after rainfall of duration stated C_1	Period of concentration, minutes																							
		180		150		135		120		105		90		75		60		45		30		20		10	
		P	C_1P	P	C_1P	P	C_1P	P	C_1P	P	C_1P	P	C_1P	P	C_1P	P	C_1P	P	C_1P	P	C_1P	P	C_1P	P	C_1P
5	0.50	2.8	1.4	3.3	1.6	3.7	1.8	4.2	2.1	4.8	2.4	5.6	2.8	6.7	3.4	8.4	4.2	11.2	5.6	16.8	8.4	25.2	12.6	50.4	25.2
10	0.60	2.8	1.7	3.4	2.0	3.8	2.3	4.2	2.5	4.8	2.9	5.6	3.4	6.7	4.0	8.4	5.0	11.2	6.7	16.8	10.1	25.2	15.1	49.6	29.8
15	0.70	2.8	2.0	3.3	2.3	3.7	2.6	4.2	2.9	4.8	3.4	5.6	3.9	6.7	4.7	8.4	5.9	11.2	7.8	16.8	11.8	25.6	17.9		
20	0.80	2.8	2.2	3.4	2.7	3.7	3.0	4.2	3.4	4.8	3.8	5.6	4.5	6.8	5.5	8.4	6.7	11.2	9.0	16.9	13.5	24.0	19.2		
25	0.825	2.8	2.3	3.4	2.8	3.8	3.1	4.2	3.5	4.8	3.9	5.6	4.6	6.7	5.5	8.4	6.9	11.2	9.2	17.7	14.6				
30	0.85	2.8	2.4	3.3	2.8	3.7	3.1	4.2	3.6	4.8	4.1	5.6	4.8	6.7	5.7	8.4	7.1	11.3	9.6	15.0	12.7				
35	0.867	2.8	2.4	3.4	2.9	3.7	3.2	4.2	3.6	4.8	4.2	5.6	4.9	6.8	5.9	8.4	7.3	11.6	10.1						
40	0.883	2.8	2.5	3.4	3.0	3.8	3.4	4.2	3.7	4.8	4.2	5.6	4.9	6.7	5.9	8.5	7.5	13.3	11.7						
45	0.90	2.8	2.5	3.3	3.0	3.7	3.3	4.2	3.8	4.8	4.3	5.6	5.0	6.8	6.1	8.7	7.8	7.8	7.0						
50	0.917	2.8	2.6	3.4	3.1	3.7	3.4	4.2	3.8	4.8	4.4	5.6	5.1	6.8	6.2	9.0	8.3								
55	0.933	2.8	2.6	3.4	3.2	3.8	3.5	4.2	3.9	4.8	4.5	5.6	5.2	6.8	6.3	10.6	9.9								
60	0.95	69.2	65.7	63.0	59.8	58.9	56.0	53.8	51.1	47.2	44.8	38.4	36.5	25.8	24.5	4.4	4.2								
75	0.95																								
90	0.95																								
105	0.95																								
120	0.95																								
135	0.95																								
150	0.95																								
180	0.95																								
Average coefficient in per cent = ΣC_1P......		90.3		89.2		88.7		87.9		86.9		85.6		83.7		80.8		76.7		71.1		64.8		55.0	

TABLE 97.—AVERAGE RUNOFF COEFFICIENTS FOR PERVIOUS SURFACES IN CLAYEY SOILS, RECTANGULAR AREAS IN WHICH THE LENGTH IS FOUR TIMES THE BREADTH. COMPUTED FROM BASIC COEFFICIENTS IN TABLE 95, COMBINED ACCORDING TO THE ZONE PRINCIPLE

In this table P represents the percentage of the total tributary area

Duration of rain on zone in min.	Run-off coefficient for one zone after rainfall of duration stated C_1	Period of concentration-minutes																							
		180		150		135		120		105		90		75		60		45		30		20		10	
		P	C_1P	P	C_1P	P	C_1P	P	C_1P	P	C_1P	P	C_1P	P	C_1P	P	C_1P	P	C_1P	P	C_1P	P	C_1P	P	C_1P
5	0.10	2.8	0.3	3 3	0.3	3.7	0.4	4.2	0.4	4.8	0.5	5.6	0.6	6.7	0.7	8.4	0.8	11.2	1.1	16.8	1.7	25.2	2.5	50.4	5.0
10	0.20	2.8	0.6	3.4	0.7	3.8	0.8	4.2	0.8	4.8	1.0	5.6	1.1	6.7	1.3	8.4	1.7	11.2	2.2	16.8	3.4	25.2	5.0	49.6	9.9
15	0.30	2.8	0.8	3.3	1.0	3.7	1.1	4.2	1.3	4.8	1.4	5.6	1.7	6.7	2.0	8.4	2.5	11.2	3.4	16.8	5.0	25.6	7.7		
20	0.35	2.8	1.0	3.4	1.2	3.7	1.3	4.2	1.5	4.8	1.7	5.6	2.0	6.8	2.4	8.4	2.9	11.2	3.9	16.9	5.9	24.0	8.4		
25	0.375	2.8	1.0	3.4	1.3	3.8	1.4	4.2	1.6	4.8	1.8	5.6	2.1	6.7	2.5	8.4	3.2	11.2	4.2	17.7	6.6				
30	0.48	2.8	1.1	3.3	1.3	3.7	1.5	4.2	1.7	4.8	1.9	5.6	2.2	6.7	2.7	8.4	3.4	11.3	4.5	15.0	6.0				
35	0.417	2.8	1.2	3.4	1.4	3.7	1.5	4.2	1.8	4.8	2.0	5.6	2.3	6.8	2.8	8.4	3.5	11.6	4.8						
40	0.435	2.8	1.2	3.4	1.5	3.8	1.7	4.2	1.8	4.8	2.1	5.6	2.4	6.7	2.9	8.5	3.7	13.3	5.8						
45	0.45	2.8	1.3	3.3	1.5	3.7	1.7	4.2	1.9	4.8	2.2	5.6	2.5	6.8	3.0	8.7	3.9	7.8	3.5						
50	0.467	2.8	1.3	3.4	1.6	3.7	1.7	4.2	2.0	4.8	2.3	5.6	2.6	6.8	3.2	9.0	4.2								
55	0.483	2.8	1.3	3.4	1.6	3.8	1.8	4.2	2.0	4.8	2.3	5.6	2.7	6.8	3.3	10.6	5.1								
60	0.50	2.8	1.4	3.3	1.7	3.7	1.8	4.2	2.1	4.8	2.4	5.7	2.8	7.0	3.5	4.4	2.2								
65	0.508	2.8	1.4	3.4	1.7	3.7	1.9	4.2	2.1	4.8	2.4	5.8	3.0	7.8	3.9										
70	0.517	2.8	1.4	3.4	1.8	3.8	2.0	4.2	2.2	4.9	2.5	5.8	3.0	8.2	4.2										
75	0.525	2.8	1.5	3.3	1.7	3.7	1.9	4.2	2.2	5.0	2.6	6.1	3.2	2.8	1.4										
80	0.533	2.8	1.5	3.4	1.8	3.7	2.0	4.3	2.3	5.0	2.7	7.2	3.8												
85	0.542	2.8	1.5	3.4	1.8	3.8	2.1	4.3	2.3	5.1	2.8	5 8	3.1												
90	0.55	2.8	1.5	3.4	1.9	3.8	2.1	4.4	2.1	5.4	3.0	2.0	1.1												
95	0.558	2.8	1.6	3.4	1.9	3.8	2.1	4.5	2.5	6.5	3.6														
100	0.567	2.8	1.6	3.4	1.9	3.9	2.2	4.5	2.6	4.3	2.4														
105	0.575	2.8	1.6	3.4	2.0	3.9	2.2	5.1	2.9	1.4	0.8														

110	0.583	2.8	1.6	3.4	2.0	4.0	2.3	5.5	3.2																		
115	0.592	2.8	1.7	3.4	2.0	4.2	2.5	3.3	2.0																		
120	0.600	2.9	1.7	3.6	2.2	5.1	3.1	1.1	0.7																		
125	0.608	2.9	1.8	3.8	2.3	4.3	2.6																				
130	0.617	2.9	1.8	4.0	2.5	2.6	1.6																				
135	0.625	2.9	1.8	4.7	2.9	0.9	0.6																				
140	0.633	2.9	1.8	3.5	2.2																						
145	0.642	3.0	1.9	2.1	1.4																						
150	0.650	3.1	2.0	0.7	0.5																						
155	0.658	3.3	2.2																								
160	0.667	3.9	2.6																								
165	0.675	3.4	2.3																								
170	0.683	2.4	1.6																								
175	0.692	1.5	1.0																								
180	0.700	0.5	0.3																								
Average coefficient in per cent = $\Sigma C_1 P$...... ...		52.2		49.5		47.9		46.3		44.5		42.2		39.8		37.1		33.4		28.6		23.6		14.9			

shapes, and applicable runoff coefficients may be chosen by the exercise of judgment. The majority of drainage districts served by artificial drains will be found to approximate rectangles in effective shape. Grunsky's studies of the effect of storage of water in transit were based upon a division of the drainage area into zones.

That high rates of rainfall are rarely uniform for considerable durations is shown by records from recording rain gages. The effect of retardation—storage in gutters, on surfaces, and in drains—is however, and the fact that flood waves tend to spread out while traveling along the drains, to equalize the flow somewhat so that the resulting runoff is not as irregular as the rate of precipitation. For many urban drainage areas where the time of flow to the outlet does not exceed say 45 min., the assumption of a uniform rate of rainfall for the period of concentration will give results that approximate the actual conditions at least as closely as the future character of and runoff coefficient for the district can be determined. Furthermore, it is usually impracticable to design for the maximum storm and maximum runoff conditions, because the cost of drains so designed would be prohibitive.

In the problem of designing storm-water drains, there are several very important factors which are incapable of exact determination and which must be fixed largely by judgment, giving due weight to local conditions, especially to the present and future characteristics of the areas to be drained, the rainfall, and the financial aspects of the problem. Since the assumption of a uniform rate of rainfall greatly simplifies the labor of design, it is believed to be justified in most cases.

The zone or time-contour principle may be applied to varying rates of rainfall, to determine the average rate or the rates producing maximum discharge from a given area. Such an application was used by C. N. Ross in a paper[1] read before the Brisbane Division of the Institution of Engineers, Australia, June 17, 1921. A similar application of this principle was made by John A. Rousculp.[2] The characteristics of the area used in Rousculp's example are shown in Fig. 85.

Starting with certain basic coefficients for elementary areas, such as those given in Table 95, the computation of the corresponding average coefficients of runoff according to the "zone principle" for any regular figure is simple. Details of such computations for a rectangular area having a length equal to four times the breadth and with outlet at the center of the short side are shown in Tables 96 and 97, applicable to impervious areas and pervious areas having clayey soil, respectively. Similar tables should be prepared for a sufficient number of regular areas to provide a suitable basis for comparison, from which the designer may select figures applicable to the actual drainage areas.

[1] "The Calculation of Flood Discharges by the Use of a Time-contour Plan."

[2] *Eng. News-Record*, 1927; **98**, 270.

Having made a decision as to the form and proportions of the equivalent regular area to be used and adopted the corresponding average coefficients of runoff for pervious and impervious surfaces, these may be combined in the ratio of the corresponding areas to obtain a table like Table 98.

TABLE 98.—RUNOFF COEFFICIENTS FOR RECTANGULAR AREAS IN WHICH THE LENGTH IS FOUR TIMES THE BREADTH, CONTAINING VARIOUS PERCENTAGES OF IMPERVIOUS SURFACES; COMPUTED FROM THE BASIC COEFFICIENTS GIVEN IN TABLE 95, COMBINED ACCORDING TO THE ZONE PRINCIPLE

Duration, minutes, or time of concentration, t	Per cent of impervious surfaces										
	00	10	20	30	40	50	60	70	80	90	100
10	0.149	0.189	0.229	0.269	0.309	0.350	0.390	0.430	0.470	0.510	0.550
20	0.236	0.277	0.318	0.360	0.401	0.442	0.483	0.524	0.566	0.607	0.648
30	0.287	0.329	0.372	0.414	0.457	0.499	0.541	0.584	0.626	0.669	0.711
45	0.334	0.377	0.421	0.464	0.508	0.551	0.594	0.638	0.681	0.725	0.768
60	0.371	0.415	0.458	0 502	0.546	0.590	0.633	0.677	0.721	0.764	0.808
75	0.398	0.442	0.486	0 530	0 574	0 618	0.661	0.705	0.749	0.793	0.837
90	0.422	0.465	0.509	0 552	0 596	0.639	0.682	0.726	0.769	0.813	0.856
105	0.445	0.487	0.530	0.572	0.615	0.657	0.699	0.742	0.784	0.827	0.869
120	0.463	0.505	0 546	0 588	0 629	0.671	0.713	0.754	0.796	0.837	0.879
135	0.479	0.521	0 561	0.601	0.642	0.683	0 724	0 765	0.805	0 846	0.887
150	0.495	0 535	0 574	0 614	0.654	0.694	0.733	0 773	0.813	0.852	0.892
180	0.522	0.560	0.598	0.636	0.674	0.713	0 751	0 789	0.827	0.865	0.903

EMPIRICAL METHODS FOR ESTIMATING STORM-WATER FLOW

In the earlier plans for drains and channels to carry away the water of storms, engineers based their designs largely upon their observations of the volumes of water seen coming from known areas in times of storm and upon the sizes of natural gutters or water courses with which they were more or less familiar. Later, the tributary areas, which could be accurately measured, were introduced as constants, and the estimates of runoff were based upon a given depth of precipitation over the whole district; but with further study it was learned that there is a gradual reduction in the immediate runoff per acre with an increase in the extent of the area and, accordingly, formulas were devised by which this fact was taken into account more or less empirically. Still more recently it has been recognized that differences in the rainfall, and especially in the intensity of the precipitation, have a direct influence upon the resulting storm-water flow, and other factors have been introduced into the formulas to take account of this and of the slope and dimensions of the drainage area. The result has been the gradual development of a number of empirical formulas and diagrams, by which the greatest quantity of storm water to be discharged from any given drainage area could be estimated.

Empirical Formulas.—The best known of these empirical formulas, reduced to a uniform notation, and with the introduction of a term expressing rate of rainfall (which was not originally used in all of them), are as follows:

Hawksley (London, 1857): $Q = Aci\sqrt[4]{(S/Ai)}$, in which $c = 0.7$ and $i = 1.0$.

Bürkli-Ziegler (Zurich, 1880): $Q = Aci\sqrt[4]{(S/A)}$, in which $c = 0.7$ to 0.9, and $i = 1$ to 3.

Adams (Brooklyn, 1880): $Q = Aci\sqrt[12]{(S/A^2i^2)}$, in which $c = 1.035$ and $i = 1$.

McMath (St. Louis, 1887): $Q = Aci\sqrt[5]{(S/A)}$, in which $c = 0.75$ and $i = 2.75$.

Hering (New York, 1889) $Q = ciA^{0.85}S^{0.27}$, or

$$Q = Aci\sqrt[6]{(S^{1.62}/A)} = ciA^{0.833}S^{0.27}$$

in which ci varies from 1.02 to 1.64. These two formulas give considerably different results.

Parmley (Cleveland, 1898) $Q = Aci\sqrt[6]{S^{1.5}/A)}$, in which c is between 0 and 1, and $i = 4$.[1]

Gregory (New York, 1907) $Q = AciS^{0.186}/A^{0.14}$, in which $ci = 2.8$ for impervious surfaces.

In these formulas S = average slope of the surface of the ground, in feet per thousand.

A comparison of these formulas is shown in Table 99, for slopes of 4, 10, 50 and 250 ft. per 1,000 ft. It will be seen that a very wide range of results may be obtained, depending upon the formula chosen.

Of these, the Bürkli-Ziegler and McMath formulas are still used to some extent. The others are of historical interest only, except as they may be needed for reference in connection with sewers and drains built in earlier times. Their derivation was discussed at some length in the first edition of this book.

It should be remembered that a runoff equivalent to 1 in. in depth in 1 hour from an area of 1 acre equals 1.008 cu. ft. per second.

The Use of McMath's Formula.—Of the foregoing formulas, that of McMath is probably most favorably known, and it has been widely used, often, no doubt, without careful study into its applicability. While the use of this or any similar formula is not to be recommended when sufficient information is available for the application of the rational method, yet there are cases when its use may be warranted. It is also convenient for use in rough preliminary computations, as it can be

[1] Parmley takes i as representing the intensity of rainfall for a period of 8 or 10 min., and for the Walworth Run Sewer (Cleveland) used $i = 4$ in order to provide for the most violent storms, and also for the further damage caused by the prevailing direction of the storms.

TABLE 99.—COMPARISON OF RESULTS OBTAINED BY USE OF SEVERAL EMPIRICAL FORMULAS FOR RUNOFF

Slope of surface, per 1,000	Flat slopes, $S = 4$				Flat slopes, $S = 10$				Medium slopes, $S = 50$				Very steep slopes, $S = 250$			
Area drained, acres	10	100	1,000	5,000	10	100	1,000	5,000	10	100	1,000	5,000	10	100	1,000	5,000
Formulas	Runoff, cubic feet per second															
Hawksley ($c = 0.7$, $i = 2$)	9.4	53	296	990	12	66	372	1,247	18	99	557	1,862	26	148	832	2,784
Bürkli-Ziegler ($ci = 2.7$)	21.5	121	679	2,271	27	152	854	2,855	40	227	1,278	4,270	60	340	1,909	6,384
Adams ($c = 1.035$, $i = 1$)	7.9	54	367	1,403	8	58	396	1,515	10	66	453	1,732	13	76	518	1,980
McMath ($c = 0.75$, $i = 2.75$)	17.2	108	683	2,474	21	130	820	2,972	28	179	1,131	4,100	39	247	1,561	5,659
Hering (A) ($ci = 1.6$)	16.5	117	825	3,240	21	149	1,057	4,150	33	231	1,632	6,411	50	356	2,521	9,902
Hering (B) ($ci = 1.6$)	15.8	108	734	2,790	20	138	942	3,602	31	213	1,451	5,548	48	329	2,242	8,565
Gregory ($ci = 2.8$)	26.3	190	1,378	5,499	31	226	1,634	6,520	42	304	2,204	8,796	57	410	2,963	11,867
Parmley ($c = 0.8$, $i = 4$)	30.8	210	1,431	5,472	39	264	1,800	6,880	58	395	2,692	10,300	87	591	4,024	15,390

Slope of surface, per 1,000	Flat slopes, $S = 4$				Flat slopes, $S = 10$				Medium slopes, $S = 50$				Very steep slopes, $S = 250$			
Diameter of sewer	12″	24″	4′	8′	12″	24″	4′	8′	12″	24″	4′	8′	12″	24″	4′	8′
Slope of sewer	.005	.002	.001	.0005	.005	.002	.001	.0005	.005	.002	.001	.0005	.005	.002	.001	.0005
Formulas	Area drained, acres															
Hawksley ($c = 0.7$, $i = 2$)	1.30	8.9	67	515	0.94	6.5	49.3	380	0.59	3.8	28.9	222	0.32	2.3	16.9	130
Bürkli-Ziegler ($ci = 2.7$)	0.42	3.0	22	165	0.31	2.0	16.3	122	0.18	1.3	9.6	71	0.11	0.7	5.6	42
Adams ($c = 1.035$, $i = 1$)	1.91	19.5	68	414	1.75	10.1	61.9	374	1.49	8.6	52.6	321	1.27	7.4	44.8	273
McMath ($c = 0.75$, $i = 2.75$)	0.68	4.2	28	184	0.41	3.4	22.2	146	0.36	2.2	15.0	98	0.24	1.5	9.9	65
Hering (A) ($ci = 1.6$)	0.84	4.7	28	162	0.63	3.5	20.5	121	0.38	2.1	12.3	73	0.23	1.3	7.4	57
Hering (B) ($ci = 1.6$)	0.83	4.8	29	181	0.62	3.6	21.8	133	0.37	2.1	12.9	80	0.22	1.3	7.9	47
Gregory ($ci = 2.8$)	0.50	2.7	16	91	0.41	2.2	13.0	75	0.29	1.6	9.2	53	0.21	1.1	6.5	37
Parmley ($c = 0.8$, $i = 4$)	0.38	2.2	13	81	0.28	1.6	10.1	61	0.18	1.0	6.2	38	0.11	0.6	3.8	23

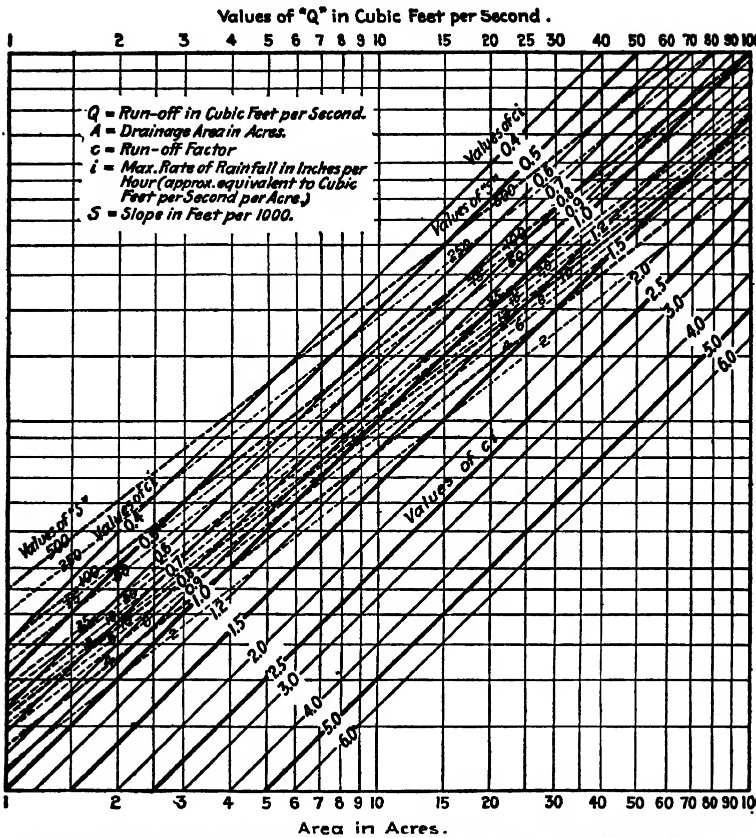


FIG. 86.—Runoff from sewered areas of 1 to 100 acres, by McMath's formula.

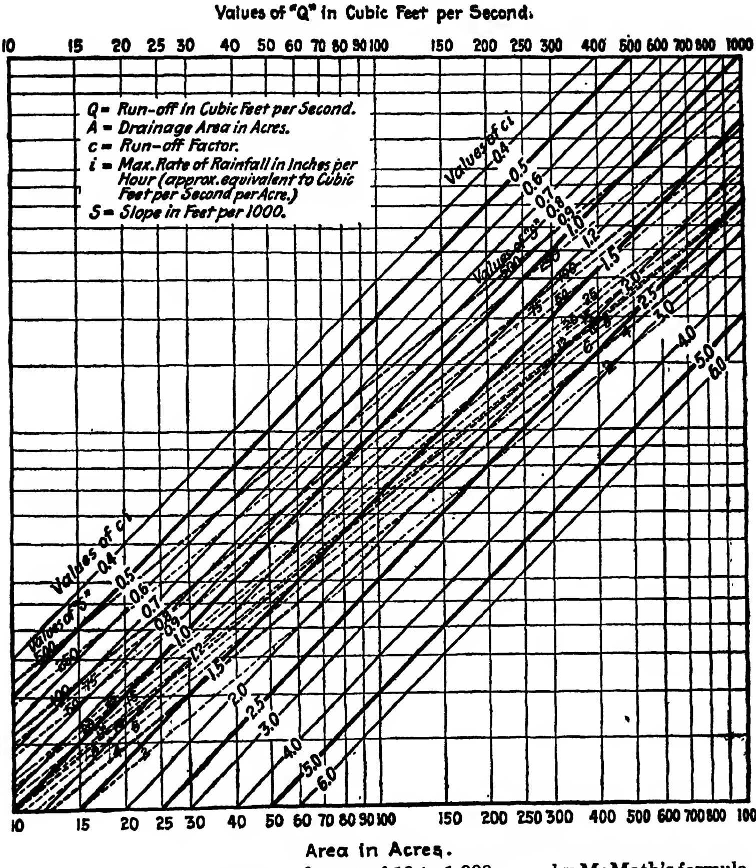


Fig. 87.—Runoff from sewered areas of 10 to 1,000 acres, by McMath's formula.

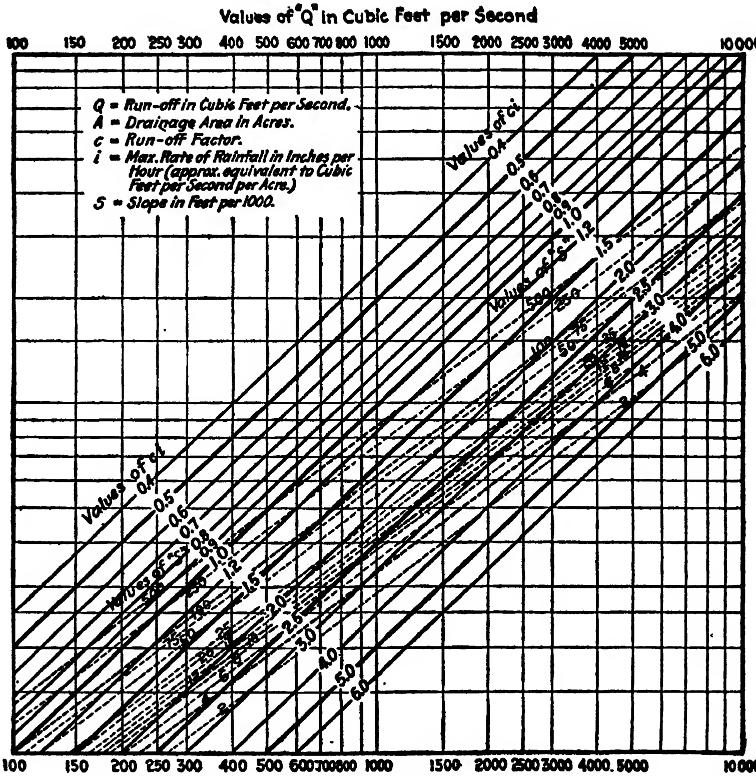


FIG. 88.—Runoff from sewered areas of 100 to 10,000 acres, by McMath's formula.

employed very rapidly by means of tables or diagrams with sufficient accuracy for such purposes, and indeed, with greater precision than the applicability of the formula warrants.

Convenient diagrams for the solution of McMath's formula are given in Figs. 86, 87, and 88. In using these diagrams, start with the given area at the bottom of the diagram and follow a vertical line to its intersection with the slope line; then follow a horizontal line to its intersection with the ci line (having first found from Table 100 or by multiplication, the product of the assumed coefficient of runoff c and the intensity of precipitation i); from this point follow a vertical line to the scale of quantities at the top of the diagram. For example, assume $A = 100$ acres, $i = 3$ in., $c = 0.70$, and $S = 15$. Then $Q = 144$ cu. ft. per second.

The values of ci for use with these diagrams are given in Table 100.

TABLE 100.—VALUES OF *ci* FOR USE WITH FIGS. 86, 87, AND 88

c	i					
	2.25	2.50	2.75	3.00	3.50	4.00
0.3	0.68	0.75	0.83	0.90	1.05	1.20
0.4	0.90	1.00	1.10	1.20	1.40	1.60
0.5	1.13	1.25	1.38	1.50	1.75	2.00
0.6	1.35	1.50	1.65	1.80	2.10	2.40
0.7	1.58	1.75	1.93	2.10	2.45	2.80
0.75	1.69	1.88	2.06	2.25	2.63	3.00
0.8	1.80	2.00	2.20	2.40	2.80	3.20
0.9	2.03	2.25	2.48	2.70	3.15	3.60

FLOOD FLOWS OF STREAMS

The foregoing discussion relates to the estimation of storm runoff for the design of storm-water drains and combined sewers. The empirical formulas have all been derived from observations of runoff from sewered areas of comparatively small extent, and their use for large areas is not justified by their source; although experience has shown that the Bürkli-Ziegler and McMath formulas are substantially as applicable to large as to small areas.

It sometimes becomes necessary, in drainage problems, to consider comparatively large areas, especially in cases where a creek passing through a city is to be converted into a covered channel.

Application of the "Rational Method" to Large Drainage Areas.—Logically, the rational method of estimating storm runoff is as applicable to large as to small drainage areas. Practically, its application to large areas is less certain than to small ones, because of the uneven distribution

of precipitation, the uncertainty as to rainfall over the entire area, the lack of definite knowledge of time of concentration, runoff coefficient, and the like. Actual records of flood flows of the stream under consideration, if available, are better than any estimation can be, provided the records cover a sufficient period of time.

It may often happen, however, that much more satisfactory information, relative to the intensity and frequency of excessive rains can be obtained than any other data bearing upon magnitude and frequency of flood flows. In such cases, it may be possible to make a better estimate by the rational method than in any other way.

Charles H. Lee, in a discussion in *Trans. Am. Soc. C. E.*, 1929; **93**, 438, quotes records of runoff of Strawberry Creek at Berkeley, California, a stream with a drainage area of 509 acres, covered by grass, brush and young pine trees, and having moderately steep slopes. Flow was measured by a weir with a recording gage, and precipitation by a recording rain gage. Time of concentration for maximum flood flows was estimated at 40 minutes. Observations did not include any severe flood flows, but discharges concentrating in 45 to 60 minutes showed runoff coefficient of approximately 0.12.

Coefficients of runoff for extreme floods are likely to be high, because such floods often come when the ground is frozen or sheathed with ice, or, in some places, when a torrential rain falls on sun-baked and largely impervious soil. The coefficient for the Scioto River (Ohio) flood of 1913, with a drainage area of 1,050 square miles and a period of concentration of about 30 hours, was found to be 0.68 for the maximum 72 hours, 0.66 for the maximum 48 hours, and 0.59 for the maximum 24 hours. The ground was water soaked and practically frozen, and other conditions were favorable for maximum runoff. In the same flood, the Miami River (Ohio), with drainage area 2,525 square miles, showed coefficients of 0.91, 0.92, and 0.76 for 72, 48, and 24 hours, respectively. The period of concentration was about 48 hours.

In estimating the frequency of flood flows of streams, it must be remembered that the simultaneous occurrence of excessive rainfall and conditions causing maximum runoff coefficient is necessary to cause a maximum flood flow, and that a combination of these conditions occurs very infrequently.

Flood-flow Formulas.—Many attempts have been made to reduce to formulas the information relating to runoff from drainage areas, so that, given the drainage area of a stream at any point, the maximum rate of discharge can be computed with reasonable accuracy. A few of these formulas are of interest and are graphically expressed in Fig. 89, in which curves of the McMath and Bürkli-Ziegler runoff formulas also are plotted for comparison.

Kuichling's Formulas.—In the report on the New York State Barge Canal (1901) Emil Kuichling, after tabulating the various records of runoff and drawing diagrams of all available flood discharge records, prepared two curves "showing the rate of maximum flood discharge on certain American and English rivers, under conditions comparable to those in the Mohawk Valley."

The formula of the first curve gives rates of discharge which may be exceeded occasionally, and is as follows:

$$Q = \frac{44{,}000}{M + 170} + 20$$

The formula of the second curve gives rates of discharge which may be exceeded rarely, and is

$$Q = \frac{127{,}000}{M + 370} + 7.4$$

This is for drainage areas of more than 100 square miles. For drainage areas less than 100 square miles in extent, Kuichling has more recently suggested the formula

$$Q = \frac{35{,}000}{M + 32} + 10$$

Kuichling has also prepared a formula for floods which may be expected to occur frequently. It is

$$Q = \frac{25{,}000}{M + 125} + 15$$

He notes that all of these formulas are intended to apply to hilly or mountainous regions, such as are found in the New England and Middle and North Atlantic States, and are probably also applicable to a rolling country having a clayey surface soil.

He has also suggested[1] as representing the maximum floods which are likely to occur in the South Atlantic States, for drainage areas not exceeding 10,000 square miles,

$$Q = \frac{41.6(620 + M)}{M + 24}$$

Murphy's Formula.—In Water Supply and Irrigation Paper No. 147 of the U. S. Geological Survey, E. C. Murphy suggests the formula

$$Q = \frac{46{,}790}{M + 320} + 15$$

Metcalf and Eddy's Formula.—The authors have suggested[2] the formula

$$Q = \frac{440}{M^{0.2688}}, \text{ or } Q = \frac{440}{M^{0.27}}$$

[1] *Trans. Am. Soc. C. E.*, 1914; **77**, 649

[2] In connection with studies for the flood-water discharge of Beargrass Creek, Louisville, Ky.

This formula gives results approximating very closely those of the Murphy formula for areas between 100 and 250 square miles, and larger results for areas beyond these limits, and is intended to represent floods which may reasonably be expected near Louisville.

Fuller's Formulas.—A very exhaustive study of flood discharge of streams, is contained in a paper on "Flood Flows," by Weston E. Fuller.[1] Fuller was the first to publish a formula in which the interval of time (corresponding to frequency of floods) appears as a factor in a formula for flood discharge, although it has been recognized for many years that the greater the interval of time, the larger the flood which is likely to occur within that time. It must be remembered that in any such study averages and probabilities are dealt with. Because a flood of a certain magnitude is likely to occur once in 100 years, it does not follow that 100 years will elapse before the occurrence of such a flood. If two floods of this magnitude should occur within 5 years, and none thereafter for 195 years, the average occurrence would still be once in 100 years.

The notation used in Fuller's formulas is:

Q = greatest 24-hour rate of runoff in a period of T years, in cubic feet per second

Q_{max} = the greatest rate of discharge during a maximum flood, in cubic feet per second

Q_{av} = the average 24-hour flood for a series of years, in cubic feet per second

T = length of period in years

M = drainage area in square miles

c = coefficient, constant for a given stream at a given point of observation

The formulas derived by Fuller from a study of all available American records are:

$$Q_{av} = cM^{0.8}$$

$$Q = Q_{av}(1 + 0.8 \log T) = cM^{0.8}(1 + 0.8 \log T)$$

$$Q_{max} = Q\left(1 + \frac{2}{M^{0.3}}\right) = cM^{0.8}(1 + 0.8 \log T)\left(1 + \frac{2}{M^{0.3}}\right)$$

In this study it is assumed that the average annual flood flow may be determined with sufficient accuracy from a record extending over a period of 10 to 15 years; in other words, that the average will not be materially affected by increasing the length of the record indefinitely.

Assuming the maximum rate of flood flow (Q_{max}) from a drainage area of 100 square miles during a period of 100 years, as unity, the corresponding maximum rates of flood flow for other areas and other periods of time would be as shown in Table 101.

[1] *Trans. Am. Soc. C. E.*, 1914; **77**, 564.

TABLE 101.—RELATION BETWEEN MAXIMUM RATES OF FLOOD FLOW PER UNIT OF AREA FROM AREAS OF VARIOUS SIZES, AND FOR PERIODS OF VARIOUS LENGTHS, ACCORDING TO FULLER'S FORMULA (WITH A CONSTANT COEFFICIENT)

Drainage area, sq. mi.	Duration of period, in years					
	1	10	50	100	500	1,000
	Relative magnitude of maximum flood discharge					
0.1	5.08	9.15	12.0	13.2	16.0	17.2
1.0	1.93	3.48	4.55	5.01	6.09	6.56
5.0	1.04	1.87	2.45	2.70	3.28	3.53
10.0	0.81	1.46	1.91	2.11	2.56	2.76
50.0	0.47	0.85	1.12	1.23	1.49	1.61
100.0	0 38	0 69	0.91	1.00	1.21	1.31
500.0	0 24	0 44	0.57	0.63	0.77	0.82
1,000.0	0.18	0 32	0.46	0.46	0.56	0.60
5,000.0	0.14	0.24	0.23	0.35	0.43	0.46
10,000 0	0.12	0.21	0.27	0.30	0.36	0.39

According to Fuller's paper, his formula expresses the general law of variation of flood flows with area and length of period. It is, nevertheless, difficult to select a proper value of *c*, unless the information available for the stream under consideration is sufficient to enable this to be computed. So many conditions may affect the value of this coefficient that it would be difficult to select a proper value, even from the extensive tables given by Fuller. The range in the coefficients computed by him is shown by Table 102.

TABLE 102.—VALUES OF THE COEFFICIENT *c* IN FULLER'S FORMULA FOR FLOOD FLOWS, FOR VARIOUS SECTIONS OF THE UNITED STATES

Section	No. of drainage areas	Values of *c*		
		Maximum	Minimum	Average
Atlantic Coast	126	140	30.0	65
St. Lawrence and Upper Mississippi	39	55	7.5	20
Ohio Basin	38	150	45.0	75
Missouri and Lower Mississippi	74	55	2.0	10
Colorado River	24	45	4.0	15
Pacific Coast	80	210	6.0	40

Fuller does not recommend any value of c for general use when information may not be available for the selection of a coefficient by comparison with some stream for which c has been computed. In his diagram showing a comparison of his formula with other formulas for flood discharge, he presents three lines representing his formula, with values of $c = 70$, $T = 100$; $c = 100$, $T = 1{,}000$; and $c = 250$, $T = 1{,}000$, respectively. It may be inferred, perhaps, that a value of $c = 100$ would be reasonable for ordinary use.

With regard to the length of the period to be used, Fuller says:

> Floods have occurred on some rivers during the last 20 years which, normally, would be repeated in not less than 1,000 years. If works are to provide for floods equal to the greatest that have been observed, a value of T of at least 1,000 should be used. Such a flood or a greater one may occur on any river at any time, but it is not likely to come soon on any particular stream. It must be remembered that the use of $T = 1{,}000$ does not mean that the corresponding flood will come at the end of 1,000 years, but that the chances are even that it will occur some time during a period of 1,000 years. It means, also, that the chances are 1 to 1,000 that it will occur in any one year, or 1 to 100 that it will occur in 10 years, or 1 to 10 that it will occur once in a century. The selection of the proper value of T then becomes a question of what chance we can afford to take.

Pettis' Formula.—Major C. R. Pettis issued in 1927 a paper entitled "A New Theory of River Flood Flow," in which he presented the formula

$$Q = 328PW^{5/4}$$

for peak flow in floods of an average frequency of 100 years, from drainage areas between 1,000 and 10,000 miles in extent, in which there are no lakes or reservoirs of sufficient consequence to materially affect the flood runoff. In this formula, P is a precipitation factor, equal to the average maximum 6-day rainfall in inches over the drainage area in a 100-year period; and W is the mean width of the area, obtained by dividing the total area by the length of the main stream from its source to the point under consideration.

By a process of reasoning similar to that employed in the "Zone Principle," Pettis reached the conclusion that the magnitude of the flood peak was a function of the width rather than the area of the drainage basin. He concluded that it would make little difference (except in the magnitude of the numerical coefficient) whether the 3- or 6-day rainfall, or that of an intermediate period, were adopted as the precipitation factor, and finally decided upon the figure for 6 days, which can be taken from an isopluvial chart (such as that given in Part V, *Tech. Rep.*, Miami Conservancy District). The numerical coefficient 328 was chosen from a study of the records of flood flows of streams in

the northeastern part of the United States from Virginia north and from Indiana east, these being the only ones for which a sufficiently long series of records are available. Pettis compared the computed and observed flood flows of 34 such streams, and found the differences to range between -12 per cent and $+9$ per cent.

This formula has not yet (1928) been sufficiently tested to warrant its general acceptance, even within the limitations of size of drainage areas set by Pettis and for the northeasterly portion of the United States. Until it has been sufficiently verified, it should be employed only as an additional check to results obtained by other methods.

Comparison of Flood-flow Formulas.—A comparison of the flood runoffs from drainage areas of various sizes, according to the various formulas for flood discharge, may be made from data in Table 103, and from the diagram in Fig. 89.

TABLE 103.—ESTIMATES BY VARIOUS FORMULAS OF FLOOD DISCHARGE OF STREAMS, IN CUBIC FEET PER SECOND PER SQUARE MILE

Formula	Drainage area, in square miles							
	1	5	10	50	100	500	1,000	10,000
Kuichling, No. 1 (occasional) $Q = \frac{44{,}000}{M + 170} + 20$	277	272	264	220	183	86	58	24
Kuichling, No. 2 (rare)	.....	..	...	..	277	153	100	19
$Q = \frac{127{,}000}{M + 370} + 7.4$ (for drainage areas of more than 100 square miles)								
$Q = \frac{35{,}000}{M + 32} + 10$ (for drainage areas of less than 100 square miles)	1,070	956	844	437				
Kuichling, No. 3 (frequent) $Q = \frac{25{,}000}{M + 125} + 15$	214	207	200	158	126	55	37	17
Murphy (Max. for N. E. U. S) $Q = \frac{46{,}790}{M + 320} + 15$	161	159	157	141	126	72	51	20
Metcalf and Eddy $Q = \frac{440}{M^{0.27}}$	440	286	237	154	127	82	68	37
McMath ($c = 0.75$; $i = 2.75$) $Q = ciA\sqrt[5]{\frac{S}{A}}$ $S = 10$	574	416	362	262	229	165	144	91
Bürkli-Ziegler ($c = 0.9$; $i = 3$) $Q = ciA\sqrt[4]{\frac{S}{A}}$ $S = 10$	611	408	344	230	193	129	109	61
Fuller: $Q_{max} = cM^{0.8}(1+0.8 \log T)\left(1 + \frac{2}{M^{0.3}}\right)$								
$c = 70$, $T = 50$	495	268	209	122	99	62	52	30
$c = 70$, $T = 100$	546	294	230	135	109	69	57	33
$c = 100$, $T = 1{,}000$	1,020	550	430	252	204	129	107	61
$c = 250$, $T = 1{,}000$	2,550	1,375	1,070	629	509	322	267	152

Effect of Snow and Ice.—It may happen in some cases that the maximum flow of streams will occur when a warm rain falls upon snow already on the ground, or when the ground may be coated with ice in such a manner as to present a practically impervious surface, as well as allowing a portion of it to melt and run off with the rain. In these cases the total runoff may amount to 100 per cent of the precipitation, or even more. In the case of streams of considerable magnitude, where the time necessary for concentration is several hours, or possibly even days, and where the maximum rate of precipitation, which probably prevailed over but a limited area, is a comparatively small factor in determining the maximum rate of runoff, maximum flood conditions are particularly likely to occur from rain falling upon snow or ice.

In such cases, it is desirable to estimate the approximate equivalent of the snow or ice upon the ground in terms of depth of water. The United States Weather Bureau "Instructions to Co-operative Observers" states that when it is impossible to measure the water equivalent of snow by melting, one-tenth of the measured depth of snow on a level open place is to be taken as the water equivalent, although it is recognized that this relation varies widely in different cases, depending on the wetness of the snow. The water equivalent of snow may be as great as one-seventh or as small as one-thirty-fourth of the depth of the snow. These figures apply to recently fallen snow; the water equivalent of snow which has been on the ground for some time and which is therefore compacted to some extent, would be greater. R. E. Horton states in the "Monthly Weather Review" (May, 1905):

> All records indicate that, for the heavy and persistent snow accumulations occurring in New York and New England, a progressive growth in the water equivalent per inch of snow on ground will usually take place as the season advances, due to compacting by wind, rain, and partial melting, and to the weight of the superincumbent mass on the lower layers. The water equivalent of compacted snow accumulation is commonly between one-third and one-fifth, or at least double that for freshly fallen snow.

The relation between the thickness of an ice layer and the corresponding depth of water is more uniform, and for practical purposes 1 in. of ice may be considered as equivalent to 0.9 in. of rain.

In the case of sewer districts, maximum run-off is much less likely to occur from rain falling upon snow or ice. Rains of great intensity are comparatively rare occurrence during the season when snow and ice are formed. Moreover, the effect of snow upon the ground would usually be to retard the flow of water, the snow acting as a sponge during the time of heaviest precipitation, and causing the runoff to be at a more gradual rate than the rainfall during this portion of the storm. It is, however, possible, under extreme conditions, that maximum runoff

might be caused by a warm rain of heavy intensity following after a period of comparatively light precipitation, by which the snow has been saturated and nearly melted, so that the maximum rate of runoff might even be in excess of the greatest rate of precipitation, and the possibility of this condition must always be borne in mind.

Records of Flood Flow of Streams.—Table 104 contains some records of flood flow of streams in the United States for streams of drainage areas less than 250 square miles in extent. The same data are shown graphically in Fig. 90.

Except as otherwise noted, these data have been taken from a paper by G. H. Matthes, entitled "Floods on Small Streams caused by Rainfall of the Cloudburst Type," and from a discussion by Harrison P. Eddy, both of which are parts of a symposium on flood problems.[1] Reference is made to these papers for the original sources of information. Mr. Matthes defines the term "small stream" as applying to

> . . . any water course in which the maximum rates of flood flow are caused, not by prolonged and widespread heavy rains, as in the case of rivers, but by downpours of exceptional intensity, of short duration, and covering areas rarely exceeding 50 square miles.

The figures tabulated have, nevertheless, been extended to include noteworthy flood flows from areas less than 250 square miles, which is as far as is warranted in a treatise devoted to sewerage problems.

Extremely high flood flows from very small drainage areas have been observed. Steep slopes and impervious soil, and sometimes snow and ice, in addition, are usually responsible for the very high rates. When it is remembered that 2,000 cu. ft. per second per square mile represents a runoff coefficient of 1.00 and a rainfall rate of 3.1 in. per hour, the rarity of such conditions, even on very small drainage areas, will be appreciated.

The designer must have in mind the extreme flood flows which have occurred and the extent of the damage which would result from inadequate provision for such floods, and make this decision as to the magnitude of flood flow to be safely handled on the basis of sound public policy. This will necessarily be a matter of judgment, in which the probable frequency of occurrence of great floods, the danger to life, and the financial conditions involved must all be given due weight.

Frequency of Floods in Streams.—An elaborate study of the relative magnitude of flood flows to be expected in various periods of time is contained in Weston E. Fuller's paper on "Flood Flows," previously referred to. According to his analysis, the greatest flood which is likely to occur in a period of T years will exceed the average annual flood by 0.8 log T times the average annual flood. The relative magnitudes of

[1] *Trans. Am. Soc. C. E.*, 1922; **85**, 1388, 1525.

TABLE 104.—FLOOD FLOW OF STREAMS

Number	Stream and locality	Drainage area in square miles	Flood flow in cubic feet per second per square mile	Date of flood
1	Beacon Brook, Fishkill, N. Y.	0.25	3,200	July 14, 1897
2	Bulls Run, Long Level, Pa.	0.58	4,170	July 15, 1914
3	Dokers Hollow, North Braddock, Pa.	0.6	4,000	June 10, 1917
4	Mann's Run, Creswell Station, Pa.	0.67	2,540	July 15, 1914
5	Mad Creek, Le Roy, N. Y.	1½	2,000 to 2,300	May, 1916
6	Green Branch, Bridgeville, Pa.	1.7	1,595	July 15, 1914
7	Arroyo, near Pueblo, Colo.	1.8	1,060	June 3, 1921
8	Cherryvale Creek, Cherryvale, Kan.	2	930	(?)
9	Indian Run, Letort, Pa.	2.1	1,930	July 15, 1914
10	Canadochly Creek, East Prospect, Pa.	2.2	1,630	July 15, 1914
11	Bull Run, Jeannette, Pa.	2.25	310+	July 5, 1903
12	[1] Moraine Run, Moraine City, Ohio	2.6	346	Sept., 1916
13	Colvin Run, Grindstone, Pa.	2.7	480	July 21, 1912
14	Starch Factory Creek, New Hartford, N. Y.	3.4	209	Sept. 3–4, 1905
15	[1] W. Fork Honey Creek, New Carlisle, Ohio.	3.5	1,000	July, 1918
16	Estanzuela River, Monterrey, Nueva Leon, Mexico	3.5	825	Aug 27–28, 1909
17	Hulls Gulch, Boise, Idaho	5	1,000	July 24, 1913
18	Mad Brook, Sherburne, N. Y.	5.0	262	Sept. 3–4, 1905
19	Breakneck Run, Bullskin Township, Pa.	5.2	(310 (250	May 19, 1902) Aug. 19, 1912)
20	Brush Creek, Jeannette, Pa.	6	500+	July 5, 1903
21	[2] Hogan's Gulch, near Eden, Colo.	6.1	1,580	
22	East Fork, Honey Creek, New Carlisle, Ohio	6.7	2,210	July 29, 1918
23	Blue Ribbon Creek, near Pueblo, Colo.	6.7	1,360	June 3, 1921
24	Cameron Arroyo, near Pueblo, Colo.	7.3	1,900	June 3, 1921
25	Osteen Arroyo, near Pueblo, Colo.	7.8	1,160	June 3, 1921
26	Burgoon's Run, near Altoona, Pa.	8.1	400+	May 20, 1894
27	Arroyo, Indiola, N. M.	8.87	1,105	July 19, 1915
28	Mill Brook, Edmeston, N. Y.	9.4	241	Sept. 3–4, 1905
29	Spring Creek, Harrisburg, Pa.	11.6	238	Feb. 15, 1908
30	East Fork, Honey Creek, New Carlisle, Ohio	11.8	1,285	July 29, 1918
31	[1] Eaton Wash, Los Angeles, Calif.	12.5	367	1914
32	Mill Creek, Erie, Pa.	12.9	1,000	Aug. 3, 1915
33	Manhan River, Holyoke, Mass.	13.0	182	Feb. 13, 1900
34	Connoquenessing Creek, Oakland, Pa.	13.6	315	Aug. 28, 1903
35	Panther Creek, Iowa	14	520	June 10, 1905
36	Rocky Creek, near Ellisville, Miss.	15	1,110	May 7, 1882
37	Arroyo, near Pueblo, Colo.	15.8	619	June 3, 1921
38	Alazan Creek, San Antonio, Tex.	16.9	1,950	Sept. 9, 1921
39	Little Devil's Creek, Iowa	19	560	June 10, 1905
40	Martinez Creek, San Antonio, Tex.	19.6	1,223	Sept. 9, 1921
41	[1] Ridley Creek, Sycamore Mills, Pa.	20.0	750	August, 1843
42	Willow Creek, Heppner, Ore.	20	1,800	June 14, 1903
43	Chase Creek (tributary of Gila River) Arizona	20	647	December, 1906

[1] *Eng. News-Record*, June, 1922; **88**, 1073.

[2] *Water Supply Paper* **487**, 40.

TABLE 104.—FLOOD FLOW OF STREAMS.—(*Continued*)

Number	Stream and locality	Drainage area in square miles	Flood flow in cubic feet per second per square mile	Date of flood
44	Cane Creek, Bakersville, N. C.	22	1,341	May 20, 1901
45	[1] Crum Creek, Delaware Co., Pa.	22	410	August, 1843
46	Apache Creek, San Antonio, Tex.	22	704	Sept. 9, 1921
47	Dry Run, Decorah, Iowa	22.3	720	Mar. 15, 1919
48	[1] River des Peres, St. Louis, Mo.	23.4	410	August, 1915
49	Trout Brook, Brooksport, N. Y.	25	158	
50	Pequonnock River, Bridgeport, Conn.	25	157	July 29–30, 1905
51	Boggs Creek, near Pueblo, Colo.	26.5	582	June 3, 1921
52	[3] Olmos Creek, San Antonio, Tex.	26.8	1,174	Sept. 9, 1921
53	Spring Creek, above Dayton, Ohio	27	210	Mar. 23–27, 1913
54	Donnels Creek, above Dayton, Ohio	27	147	Mar. 23–27, 1913
55	Pinal Creek, Globe, Ariz.	30	440	Aug. 17, 1904
56	Peck Creek, near Pueblo, Colo.	34.4	564	June 3, 1921
57	Sawkill River, Kingston, N. Y.	35	228	April, 1895, also 1896
58	Turtle Creek, above Dayton, Ohio	35	175	Mar. 23–27, 1913
59	Lake Roland, Maryland	39	230	1868
60	San Antonio River, San Antonio, Tex	32.4	960 to 1,200	Sept. 9, 1921
61		41	580	Sept. 9, 1921
62		45	333	Sept. 9, 1921
63	Cameron Creek, Hurley, N. M.	44	125	Aug. 14, 1913
64	Elkhorn Creek, Keystone, W. Va.	44	1,363	June 22, 1901
65	Sixmile Creek, Ithaca, N. Y.	46	195	June 21, 1905
66	[1] Darby Creek, Delaware Co., Pa.	48	580	August, 1843
67	Pine Creek, Paris, Tex	48	320 to 410	May 12, 1920
68	Little Conemaugh River, Johnstown, Pa.	48.6	206	May 30–31, 1889
69	Lost Creek above Dayton, Ohio	52	571	Mar. 23–27, 1913
70	Santa Ysabel Creek, near Mesa Grande, Calif.	53.4	395	Jan. 27, 1916
71	Tawawa Creek, above Dayton, Ohio	54	239	Mar. 23–27, 1913
72	Rock Creek, near Pueblo, Colo.	59	913	June 3, 1921
73	[1] Chester Creek, Bridgewater, Pa.	62	1,000	Aug., 1843
74	Ludlow Creek, above Dayton, Ohio	65	266	Mar. 23–27, 1913
75	Dry Creek, Pueblo, Colo.	70	300	June 2, 1921
76	[3] San Antonio River, San Antonio, Tex.	85	499	Sept. 9, 1921
77	Gallinas River, Las Vegas, N. M.	89	131	Sept. 29, 1904
78	Putah River, Guenoc, Calif.	91	270	Mar. 10, 1904
79	North Fork Creek, Brookville, Pa.	97	124	July 17, 1912
80	Otay River, Lower Otay, Dam, Calif.	98.6	379	Jan. 27, 1916
81	San Jacinto River, San Jacinto, Cal.	108	278	Jan., 1916
82	Santa Ysabel Creek, Ramona, Calif.	110	258	Jan. 27, 1916
83	Devil's Creek, Iowa	143	600+	June 10, 1905
84	[3] Salado Creek, Salado, Tex.	148	966	Sept. 10, 1921
85	Mora River, La Cueva, N. M.	159	140	Sept. 29, 1904
86	Sweetwater River, SweetWater Dam, Calif.	181	251	Jan. 27, 1916
87	Cave Creek, Phoenix, Ariz.	200	125	Aug. 21, 1921
88	San Luis Rey River, Mesa Grande, Calif.	209	280	Jan. 18, 1916

[3] *Water Supply Paper* 488, 14, 19.

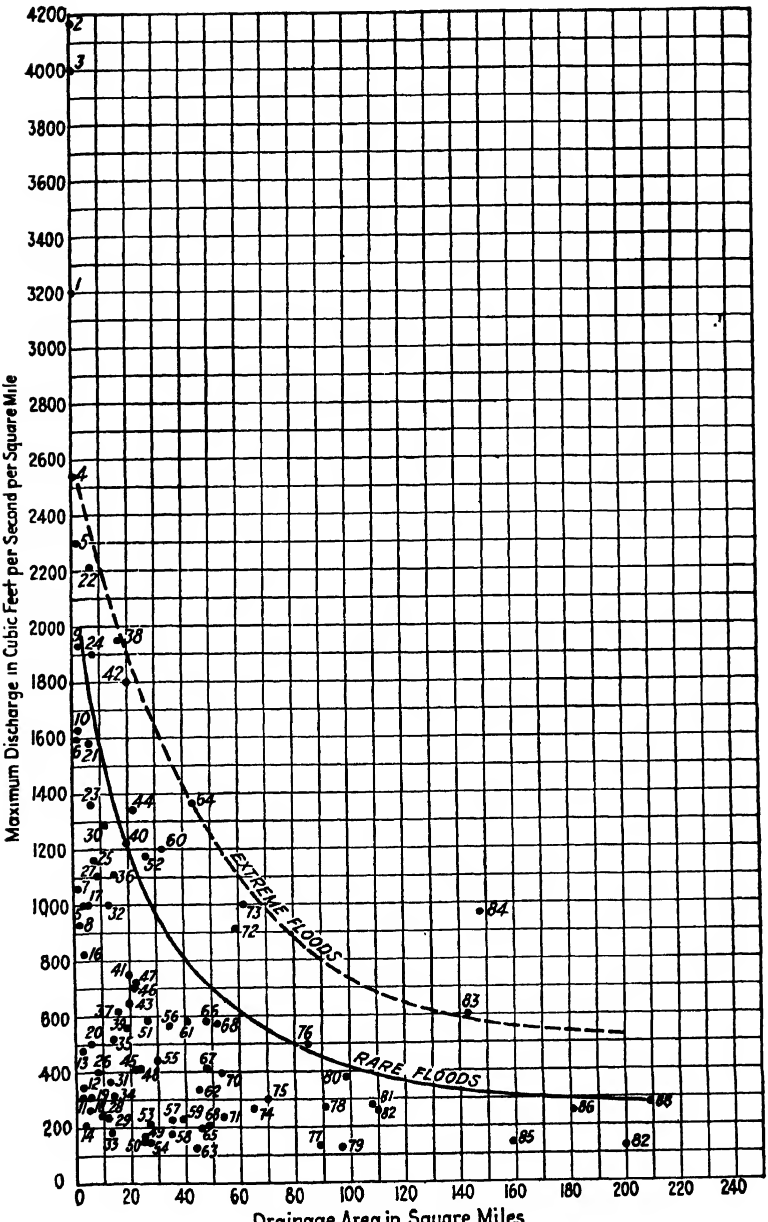


Fig. 90.—Maximum flood flows from small drainage areas.

floods which will probably recur in periods of various durations according to this relation, are shown in Table 101.

Robert E. Horton, in *Bull. Z* of the U. S. Weather Bureau, on "The Floods of 1913," has explained the application of the mathematical theory of probabilities to the estimation of the probable recurrence of floods of various magnitudes, and derived the formula

$$T = \left(\frac{Q + 50{,}000}{80{,}000}\right)^{7.14}$$

TABLE 105.—AVERAGE INTERVAL BETWEEN FLOODS OF VARIOUS MAGNITUDES IN SOME AMERICAN RIVERS

River	Drainage area, shed, sq. mi.	Length of record, years	Maximum observed flood, c.f.s. per sq. mi.	Magnitude of flood as compared to maximum flood			
				0.6 to 1.0	0.7 to 1.0	0.8 to 1.0	0.9 to 1.0
				Average frequency, years			
Kennebec...	4,380	12	25.4	2	4	12	12
Androscoggin....	2,320	12	23 8	4	6	12	12
Merrimac	4,553	59	18.0	...	....	15	20
Connecticut.....	10,234	105	20 0	..	....	12	53
Hudson.......	4,500	35	15.6	...	....	18	35
Genesee........	2,428	119	19–22	...	....	60	119
Passaic.........	8.227	26	42.5	...	13	26	26
Raritan.........	806	96	64.5	...	24	48	96
Delaware........	6,855	120	37.1	...	....	40	120
Susquehanna.....	24,030	17	28–30.6	...	6	9	17
Cape Fear.......	3,860	15	18–23	0.4	0.6	2	7
Savannah.. . .	7,500	66	40	0.8	1.5	5.5	33
Alabama...... ..	15,400	14	9.5	0.6	0.7	1.6	5
Black Warrior. .	4,900	17	32	0.4	0.5	0.9	2
Monongahela....	5,430	20	38.1	5	10	20	20
Youghiogheny....	782	32	54–59	4	16	32	32
Allegheny........	9,220	31	26.7	0.3	1	3	8
Ohio............	23,800	22	20.8	2	3	10	22
Illinois..........	15,700	16	3.3	...	2	3	5
Rio Grande......	28,067	11	1.2	4	6	6	11
Colorado........	37,000	9	3.3	...	5	9	9
Arkansas........	4,600	10	2.4	3	5	10	10
Bear............	6,000	15	1.8	3	5	15	15

for the Hudson River at Mechanicsville, where the drainage area is 4,500 square miles. In this formula T = average period of recurrence in years and Q = maximum flood flow in cubic feet per second. In a discussion upon Fuller's paper on "Flood Flows" Horton also gives[1] the general formula

$$T = \sqrt[4]{\frac{QM}{4{,}021.5}}$$

TABLE 106.—FREQUENCY OF FLOODS WHICH CAUSED SERIOUS DAMAGES ON SEVERAL AMERICAN STREAMS

Stream	Location	Dates of serious floods	Inter-val, years	Average interval, years
Mill Creek...........	Erie, Pa.	1878		
		1893	15	
		1915	22	18
Willow Creek...........	Heppner, Ore.	1883		
		1903	20	
Jones Falls............	Baltimore, Md.	1754		
		1786	32	
		1817	31	
		1837	20	
		1842	5	
		1858	16	
		1860	2	
		1868	8	
		1887	19	17
Cherry Creek...........	Denver, Colo.	1864		
		1878	14	
		1885	7	
		1912	37	18
Codorus Creek..........	York, Pa.	1744		
		1758	14	
		1786	28	
		1817	31	
		1821	4	
		1847	26	
		1850	3	
		1834	34	
		1884	0	18

[1] *Trans. Am. Soc. C. E.*, 1914; **77**, 665.

derived from 20-year records of Neshaminy, Perkiomen, and Tohickon Creeks, near Philadelphia.

Information indicating the relative frequency of floods of various magnitudes on 23 American rivers are given by E. C. Murphy in Water Supply and Irrigation Paper 162 of the U. S. Geological Survey. The most significant information, compiled from his records, is given in Table 105.

In the article previously referred to, Matthes[1] gives the records of floods which caused serious damages on several small streams, which are summarized in Table 106.

The Department of Public Works of the State of California, in Appendix *A* of its report for 1923, printed a series of curves showing the magnitude and occurrence of flood discharges of the rivers of that state, extended to indicate the probable frequency in very long terms of years.

C. H. Pierce[2] has brought together the most important data relating to floods on New England Streams for which continuous records have been kept; but most of these relate to comparatively large streams.

[1] *Trans. Am. Soc. C. E.*, 1922; **85**, 1396.
[2] *Jour. Boston Soc. C. E.*, 1924; **11**, 327.

CHAPTER IX

STORM DRAINS AND COMBINED SEWERS

Definitions.—A *storm drain* is a conduit for carrying off surface water and storm water.

A *combined sewer* is a sewer intended to receive domestic sewage, industrial wastes, and surface and storm water. In the combined system, the rate of discharge of domestic sewage and industrial wastes is so small compared to the rate of runoff of storm water (usually being less than the error in estimating storm water) that the former is generally neglected and the sizes are determined from the estimate of storm flow alone.

DESIGN OF COMBINED SEWERS OR STORM DRAINS

The design of combined sewers or storm drains requires, first, the determination of the storm runoff as described in the preceding chapter, and, second, the selection of the proper dimensions for the sewer, taking into account the available slope and other topographical and physical conditions.

Storm Runoff.—The determination of storm runoff or required capacity of the sewer may be arrived at either by the rational method or by the use of an empirical formula. In this discussion, the rational method will be followed. The basic principles of design are the same, whichever method is used.

The computation of the runoff requires the determination of the following basic data:

1. The time-intensity rainfall curve to be used as a basis of design.
2. The probable future condition of the drainage area, *i.e.*, the percentage of impervious surface which may be expected when the district shall have been developed to the extent assumed.
3. The runoff coefficient, *i.e.*, the proportion of the rainfall which will run off over the surface of the ground.
4. The probable time required for water to flow over the surface of the ground to the first inlet, called the "inlet time" or "time of entrance."
5. The area tributary to the sewer at the point at which the size is to be determined.

6. The time required for water to flow in the sewer from the first inlet to the above-mentioned point, which added to the inlet time, gives the time of concentration.

Then by the application of the proper runoff coefficient to the rainfall for the time of concentration, the rate of runoff per unit of area may be computed.

Adoption of Rainfall Curve.—The form of the rainfall curve will be determined by the records of excessive rainfall for the locality. The particular frequency curve to be adopted will depend largely upon economic conditions, and the extent to which it is necessary and financially practicable to avoid occasional surcharging of the sewers and possible damage from flooding. Something will also depend upon the relative position of the rainfall curves of different frequencies; thus, if the curve of 15-year frequency lies but little below the 25-year curve, there will be little saving in cost by adopting the former, and it may be better to adopt the 25-year than the 15-year curve as the basis of design. Perhaps a rainfall curve of 15-year frequency is likely to be adopted more often than any other. In many cases, economic considerations may not justify the construction of drains for flows likely to occur less often than once in 5 years.

Determination of Runoff Coefficients.—An estimate must be made of the probable coefficient of imperviousness for each sewer district as it is likely to exist at the end of the assumed economic period of design, say 40 years for instance. Such an estimate can be made with greater certainty where a proper zoning ordinance is in force than elsewhere, although too much credence must not be given to the permanent effect of such an ordinance. Suitable runoff coefficients for impervious and pervious surfaces must then be assumed, bearing in mind the character of the earth and other local conditions which may have an effect. These coefficients may then be combined, either directly or according to the zone principle, and modified if desired, for the effects of distribution of rainfall, retardation, and retention, thus obtaining the runoff coefficients applicable to the locality under consideration at the end of the period of design.

Economic Considerations.—It must be remembered that runoff coefficients are chosen to represent conditions as they are expected to be in the future, and, consequently, the frequency adopted for design will represent the average interval between occasions when the capacity of the drain will be equalled or exceeded after the district has developed to the assumed condition. During the period of development, the flood conditions would be much more rare, if experienced at all.

This consideration may be illustrated by an example. Assume that a drain is to be built to serve an area for which the time of concentration

is 45 min. and that the rates of rainfall corresponding to various frequencies and this time of concentration are:

40 years, 2.30 in. per hour
20 years, 2.10 in. per hour
15 years, 2.00 in. per hour
10 years, 1.90 in. per hour
5 years, 1.70 in. per hour

Now, if the assumed runoff coefficient when the district has developed be 0.60, and the estimated coefficient at present be 0.40; if the 15-year rainfall curve is to be used; and if it be further assumed that the district develops uniformly, reaching full development (corresponding to the coefficient 0.60), in 20 years; then:

1. The drain will be designed for a runoff equivalent to $0.60 \times 2.00 = 1.20$ in. in depth per hour.

2. This rate of runoff corresponds to a present rate of rainfall of $1.20/0.40 = 3.00$ in. per hour, which for a 45-min. period, has a probable frequency of about once in 1,000 years.[1]

3. The corresponding rate of rainfall required to fill the drain after 10 years, when the coefficient of runoff is 0.50, is 2.40 in. per hour, and the probable frequency of such a rain is once in 100 years.[1]

It is obvious that the drain is very unlikely to be filled to capacity at any time within 15 years after construction. After 20 years it may be expected that it will be taxed to its capacity or surcharged at average intervals of 15 years.

This being the case, it may be questionable whether the construction of so large a drain would be justified, particularly as the assumed degree of development may never be experienced. The basis of design should be re-examined and it may be found wiser to design for a condition of development which will be reached at an earlier date, even if still further development goes on thereafter; or a rainfall curve of 10- or 5-year frequency may be adopted. If the basis of design is so modified, it may be expected that at some future time it will become necessary to provide a relief sewer, but this may be more economical than to build the larger structure at once.

Runoff Curves.—Having determined upon the rainfall curve and runoff coefficients to be used, these may advantageously be combined and a series of runoff curves prepared, similar to those shown in Fig. 91 for Detroit, Mich. The data from which these curves were constructed are given in Table 107.

A similar set of curves devised by Darwin A. Townsend for use at Milwaukee, Wis., is shown in Fig. 92.[2] The rainfall curve was derived

[1] Estimated by plotting on probability paper the rates and frequencies taken as basic data.

[2] *Eng. News-Record*, 1922; 89, 1039.

from 17 years' records, and presumably may be taken as representative of a 15-year frequency. As shown, it corresponds somewhat closely with a curve the equation of which is $i = \frac{136}{t + 20}$. The runoff curves were derived directly by the application of the coefficients indicated, the zone principle not being employed. As indicated, the rates of runoff for periods less than 15 min. have been taken as uniform for each value of

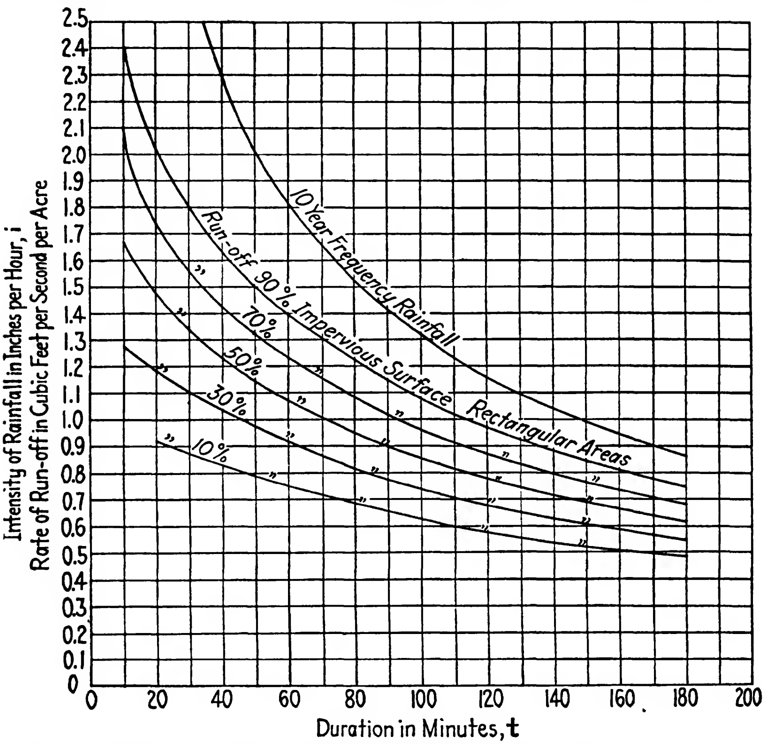


FIG. 91.—Runoff curves for Detroit, Mich., based upon rainfall curve of 10-year frequency and various percentages of impervious surfaces.

Computed for rectangular area with length = 4 × breadth, from basic coefficients in Table 95, combined according to "zone principle."

the coefficient. A coefficient of 0.35 has been adopted for use in residential sections of Milwaukee.

Time of Concentration.—As a preliminary to computing the time of concentration, the inlet time to be used must be determined. Theoretically, this is the time of concentration at the inlet, or the time required for water to flow from the most distant point to the inlet. Practically, it includes the effect of various retarding influences, especially the time

TABLE 107.—RATES OF RUNOFF IN CUBIC FEET PER SECOND PER ACRE, COMPUTED FROM RAINFALL CURVE FOR 10-YEAR FREQUENCY AT DETROIT, FOR VARIOUS PERCENTAGES OF IMPERVIOUS SURFACE. COMPUTATIONS BASED UPON RECTANGULAR AREAS IN WHICH LENGTH EQUALS FOUR TIMES BREADTH; COEFFICIENTS COMBINED ACCORDING TO ZONE PRINCIPLE

Period of concentration, minutes t	Intensity of precipitation, inches per hour i	Per cent of impervious surfaces																					
		00		10		20		30		40		50		60		70		80		90		100	
		C	Ci	C	Ci	C	Ci	C	Ci	C	Ci	C	Ci	C	Ci	C	Ci	C	Ci	C	Ci	C	Ci
10	4.77	0.149	0.71	0.189	0.90	0.229	1.09	0.269	1.28	0.309	1.47	0.350	1.67	0.390	1.86	0.430	2.05	0.470	2.24	0.510	2.43	0.550	2.62
20	3.31	0.236	0.78	0.277	0.92	0.318	1.05	0.360	1.19	0.401	1.33	0.442	1.46	0.483	1.60	0.524	1.73	0.566	1.87	0.607	2.01	0.648	2.14
30	2.69	0.287	0.77	0.329	0.88	0.372	1.00	0.414	1.11	0.457	1.23	0.499	1.34	0.541	1.45	0.584	1.57	0.626	1.68	0.669	1.80	0.711	1.91
45	2.16	0.334	0.72	0.377	0.81	0.421	0.91	0.464	1.00	0.508	1.10	0.551	1.19	0.594	1.28	0.638	1.38	0.681	1.47	0.725	1.59	0.768	1.66
60	1.82	0.371	0.67	0.415	0.75	0.458	0.83	0.502	0.91	0.546	0.99	0.590	1.07	0.633	1.15	0.677	1.23	0.721	1.31	0.764	1.39	0.808	1.47
75	1.59	0.398	0.63	0.442	0.70	0.486	0.77	0.530	0.84	0.574	0.91	0.618	0.98	0.661	1.05	0.705	1.12	0.749	1.19	0.793	1.26	0.837	1.33
90	1.41	0.422	0.60	0.465	0.66	0.509	0.72	0.552	0.78	0.596	0.84	0.639	0.90	0.682	0.96	0.726	1.02	0.769	1.08	0.813	1.15	0.856	1.21
105	1.28	0.445	0.57	0.487	0.62	0.530	0.68	0.572	0.73	0.615	0.79	0.657	0.84	0.699	0.89	0.742	0.95	0.784	1.00	0.827	1.06	0.869	1.11
120	1.16	0.463	0.54	0.505	0.59	0.546	0.63	0.588	0.68	0.629	0.73	0.671	0.78	0.713	0.82	0.754	0.87	0.796	0.92	0.837	0.97	0.879	1.02
135	1.07	0.479	0.51	0.521	0.56	0.561	0.60	0.601	0.64	0.642	0.69	0.683	0.73	0.724	0.77	0.765	0.82	0.805	0.86	0.846	0.91	0.887	0.95
150	0.98	0.495	0.48	0.535	0.52	0.574	0.56	0.614	0.60	0.654	0.64	0.694	0.68	0.733	0.72	0.773	0.76	0.813	0.80	0.852	0.83	0.892	0.87
180	0.87	0.522	0.45	0.560	0.49	0.598	0.52	0.636	0.55	0.674	0.59	0.713	0.62	0.751	0.65	0.789	0.69	0.827	0.72	0.865	0.75	0.903	0.79

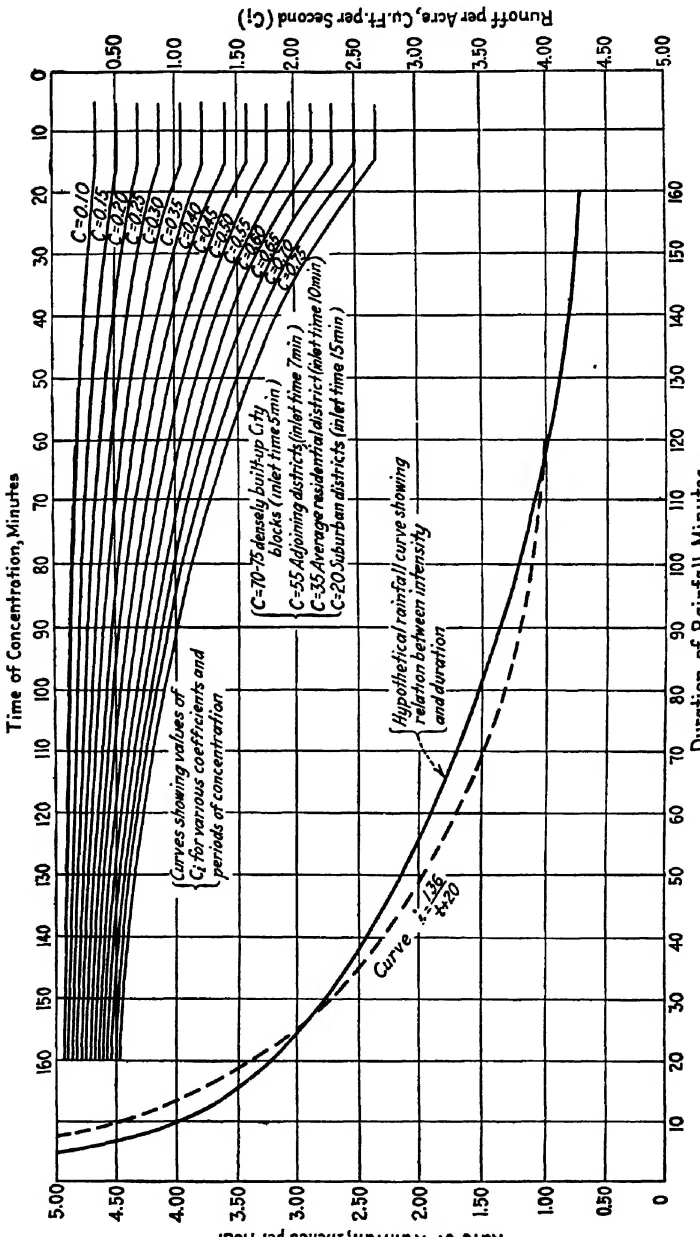


FIG. 92.—Curves for design of storm drains at Milwaukee.

required to pass through the inlet structure to the sewer or drain. Very few inlets or catch basins now in use are well fitted for the rapid passage of large quantities of water. Allowance must be made, however, for such improvements as are likely to be made, and the adopted inlet time should not necessarily be taken as that required with present gutters and inlets.

Data from Maps and Profiles.—The remaining data for the computations must be taken from an accurate plat of the district, similar to Fig. 93. On this are entered the elevations of the proposed or establhised street and alley grades and, if no contour map has been made, the existing surface elevations should also be shown. The storm-water inlets along the gutters of the streets and alleys are then located on the plat on the higher side of all street intersections and at all low points between streets, usually with no interval greater than 600 or 700 ft. between inlets. After the locations of the inlets have been fixed on the map, sewers to reach them are laid out, attention being paid to the sewerage of all private lots in the district. The most economical layout usually follows the natural surface slopes in the shortest line toward the outlet of the district and concentrates the storm flow as rapidly as possible. Sometimes several preliminary layouts should be made and compared to ascertain which is the cheapest.

The area tributary to the sewer at any point may now be determined. The designer must form a mental picture of the district as it will be at the end of the period of economic construction, with the grading and paving done and buildings erected. This concept is necessary in order to locate the minor ridge lines dividing the small areas draining into streets from those draining into alleys, and to fix the areas tributary to each inlet.

The final step in preparing the data is to fix in a preliminary way the slopes of the sewers. The start in this work of approximating slopes is made at the lower end where the elevation is fixed approximately by outside conditions. Then in the second trial, beginning at the upper end, the final slopes can be established at the same time the sizes are determined.

EXAMPLE OF DESIGN OF A STORM DRAIN

The district shown in Fig. 93 is a portion of the Carlisle Brook drainage area in the City of Springfield, Mass. The location of the proposed main drain (the so-called "Carlisle Brook Drain") into which the district is to be drained, is shown on the plan and the invert elevation is given at the point where the proposed branch drain is to be connected and for which provision has been made in the design of the main drain. The required minimum elevation of the invert of the branch drain is,

therefore, given at the proposed point of discharge into the Carlisle Brook drain.

A careful study of local conditions, including the present and probable future development of the district, indicates that a coefficient of imperviousness of 0.40 is reasonable, and that it is proper to assume a coefficient of runoff of $c = 0.35$. The zone principle is not applied.

The inlet time has been assumed to be 20 min.

The rate of rainfall is to be taken from the assumed curve of intensity of precipitation represented by the formula $i = 20.4/t^{0.61}$. This formula

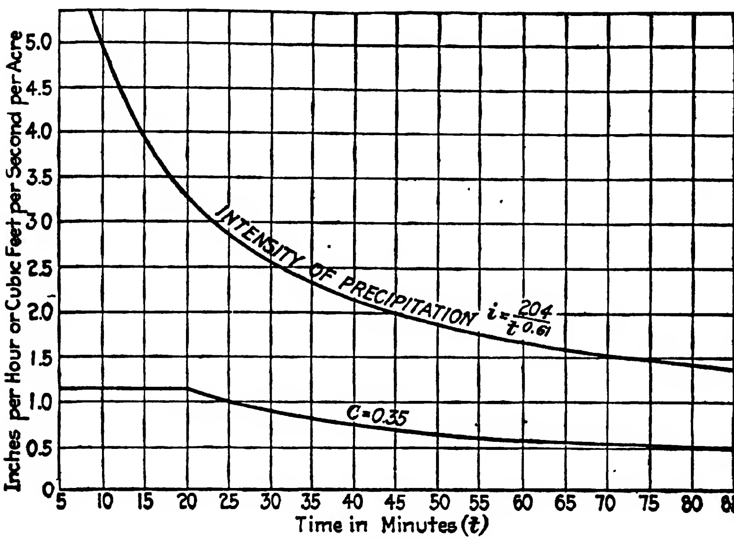


FIG. 94.—Rainfall and run-off curves for Carlisle Brook Drainage District, Springfield, Mass.

indicates the rate of rainfall which may be expected to be equalled or exceeded once in 5 years. The rainfall and runoff curves are shown on Fig. 94.

While it was recognized that drains designed on this basis might be overtaxed on the average once in 5 years, in view of other financial obligations facing it, the city was not thought to be justified in going to the extent of providing for storms of greater intensity, which would have involved greater cost. During the earlier years of the life of the drains, they will be able to care for much greater rates of flow than later, because the assumed coefficient of runoff is based upon future rather than present conditions. A progressive increase in impervious surface and runoff will be caused by the gradual substitution of roofs and paved areas for unimproved areas such as woods and grasslands as the city grows. In

the future, when the district is more densely built up and when the city has available funds, relief drains can be constructed to serve those areas where flooding has become sufficiently serious to warrant the expenditure.

Figure 93 shows the streets with surface contours, also the drainage area, within the dash-and-dot lines. The limits of this area are influenced not only by the surface contours but also markedly by the areas served by existing combined sewers and drains not shown on the plan.

The drains are to be designed, in general, with the crown at a depth of at least 5 ft. below the surface of the street.

The minimum size of drain is to be 12 in.

The assumed minimum allowable velocity is 3 ft. per second when flowing full.

The capacity of the drains is to be determined by the use of a value of n = 0.015 in the Kutter formula for sizes of 24 in. and under, and n = 0.013 for sizes over 24 in., it being assumed that the smaller sizes will be of vitrified pipe and the larger sizes of monolithic concrete with smooth interior surfaces.

The following steps must now be taken in order to reach a rational solution.

1. Draw a line to represent the drain in each street or alley to be drained. Place an arrow near each drain to show the direction of flow in it. The drains should, in general, slope with the street surface. It will usually prove to be more economical, however, so to lay out the system that the water will reach the main drain by the most direct route. Also, in general, it will prove best to concentrate the flow from small areas as quickly as possible into one drain.

In some localities with a large percentage of pervious surface, the roof water is allowed to discharge upon the ground, thence flowing to the gutter inlets and the drains. In some such instances, drains are provided only to the last gutter inlet, rather than to a point opposite the last house lot, thereby effecting some saving in cost. This practice is open to the criticism that it does not give equal service to all property and is, therefore, inequitable. In the example under consideration, it is intended to provide drainage facilities for all property within the district.

2. Locate the manholes tentatively, giving to each an identification number. In this example, a manhole is to be placed at each bend or angle, at all junctions of drains, at all points of change in size or slope and at intermediate points where the distance exceeds 400 ft., except where a good velocity will be available during practically all conditions of flow, in which case the interval may be increased to 700 or 800 ft. Sufficient manholes should be built to allow access for inspection and cleaning; later when the profiles are drawn and the final slopes fixed, it may be found desirable to change the location of some manholes in order to have the drains at the most advantageous depth, particularly where the

TABLE 108.—DESIGN OF STORM DRAINS FOR CARLISLE BROOK DISTRICT, SPRINGFIELD, MASS.

	From man-hole	To man-hole	Location, street	Length, feet	Tributary area		Time of flow		Rate of runoff from curve, cubic feet per second per acre	Total runoff, cubic feet per second	Slope of drain	Veloc-ity, feet per second	Diam-eter, inches	Capa-city, cubic feet per second	Fall, feet	Elevation of invert	
					Incre-ment, acres	Total, acres	To upper end, minutes	In section, minutes								Upper end	Lower end
	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17
1	1	2	Dawes	300	2 28	2 28	20 0	1.7	1.15	2 6	0 0049	3 0	15	3.7	1 47	199.85	198.38
2	2	3	Dawes	300	2 40	4.68	21 7	1.7	1.10	5.1	0 0037	3.0	18	5 3	1 11	198 13	197.02
3	3	4	Dawes	300	2.15	6 83	23 4	1.7	1 05	7.2	0.0028	3 0	22	7.9	0.84	196 69	195.85
4	4	5	Dawes	165	1 54	8 37	25.1	0.9	1 00	8.4	0.0031	3 2	22	8.4	0 51	195 85	195.34
5	5	6	Andrew	325	2 16	10 53	26 0	1 6	0.99	10.4	0.0029	3.3	24	10.4	0 94	195 17	194.23
6	6	7	Burr	400	3 11	13 64	27.6	2.1	0 94	12 8	0 0017	3 2	27	12.8	0 68	193 98	193.30
7	7	8	Burr	35	6 02	19 66	29 7	0 2	0.90	17 7	0 0018	3 6	30	17 7	0.06	193.05	192.99
8	8	9	Burr	230	10 14	29 80	29 9	1.1	0 90	26.8	0 0011	3.3	39	27.0	0.24	192 24	192.00
9	9	10	Burr	240	5 65	35 45	31.0	1.0	0.88	31 2	0 0014	3.8	39	31 3	0.34	192.00	191.66
10	10	11	Oak Grove Ave.	110	11 91	47.36	32 0	0 5	0 86	40 7	0.0011	3.7	45	40 7	0 12	191.16	191.04
11	11	12	Oak Grove Ave.	95	11 02	58 38	32.5	0.4	0 85	49.6	0 0012	4.0	48	49.6	0.11	190.79	190.68
12	12	13	Shattuck	295	5.60	63 98	32 9	1.1	0 84	53.8	0.0014	4.4	48	55.0	0.41	190.68	190.27
13	13	14	Shattuck	260	19 73	‘83 71	34 0	1.0	0 82	68.6	0.0012	4.4	54	70 0	0 31	189 77	189.46
14	14	15	Shattuck	145	11 30	95.01	35 0	0 6	0 81	77.0	0 0011	4.3	57	78.0	0.16	189-21	189.05
15	15	16	Shattuck	225	2 88	97.89	35.6	0.8	0 80	78.3	0 0012	4.5	57	78.5	0.26	189.05	188.79
16	16	17	Shattuck	380	13 07	110.96	36.4	1.3	0.79	87.6	0.0014	5.0	57	88.0	0.54	188.79	188.25
17	17	18	Private Land	165	4.16	115.12	37.7		0.77	88.6	0.0015	5.0	57	89.0	0.24	188.25	188.01
					115 12												

slope of the street surface is not substantially uniform. Other considerations, such as obstacles under the street, may require the installation of additional manholes, due to change in alignment of the drain or special forms of construction involved in junctions or connections with other drains.

3. Sketch the limits of the drainage areas tributary at each manhole. The assumed character of future development and the topography will determine the proper limits.

4. Measure each individual area by planimeter or other methods which will give equally satisfactory results.

5. Prepare a table in which to record the data and steps in the computations of each section of drain between manholes.

The computations for the longest line of this section are shown in Table 108.

Each lateral is then designed in a similar way. If necessary, the design of the sub-main is modified so as to serve the laterals properly.

BASIS OF DESIGN OF STORM-WATER DRAINS IN VARIOUS CITIES

No attempt has been made to include in this section any exhaustive statement of practice throughout the country, but information has been obtained (1923–1924) from a number of the large cities in which the methods employed are believed to be typical of the best practice, and it is summarized below.

Baltimore, Md.[1]—The rational method is used, estimating precipitation from the formula $i = \frac{105}{t + 10}$, and using for runoff coefficients:

0.60 to 0.70 for business areas
0.50 to 0.60 for downtown residential areas
0.40 for suburban areas
0.30 for rural areas

Inlet time is assumed according to local conditions, between 4 and 10 min.

Boston, Mass.[2]—Lovejoy's "Sewer and Runoff Diagram No. 3" is used, combining graphical solutions of the rational method of estimating runoff and of the determination of sewer size. Precipitation is estimated by the Dorr formula[3] $i = \frac{150}{t + 30}$. Coefficients of runoff are:

0.95 to 1.00 for roofs and concrete or asphalt pavements
0.70 to 0.90 for macadam streets and brick sidewalks
0.20 to 0.70 for earth surface
0.00 to 0.25 for gravel and sand

Inlet time is usually taken at 5 min.

[1] Milton J. Ruark, Engineer of Sewers.
[2] F. A. Lovejoy, Designing Engineer.
[3] Approximate frequency 10 years. (Metcalf & Eddy.)

Buffalo, N. Y.[1]—The rational method is followed. Rainfall is estimated from the equation $i = \frac{11}{t^{0.41}}$. Coefficients of runoff are

0.95 for business district
0.60 for urban residential district
0.40 for outlying or suburban districts

Assumed inlet time varies with conditions. Total time of concentration for storm drains serving areas wholly within the city ranges from 40 to 90 min.; for a large drain receiving most of its water from rural area without the city, 5½ hours.

Chicago, Ill.[2]—Both the rational method and empirical formulas are used, the former for large and important projects, the latter usually for drains serving small areas. Rainfall is estimated by the formula $i = \frac{28}{t^{0.7}}$.

Assumed coefficients of runoff are:

0.90 for loop district (center of city)
0.35 for commercial district with 65 per cent impervious surface
0.30 for apartment-house district, closely built, sandy soil
0.15 for undeveloped sandy area, zoned for industrial development

Inlet time is usually taken at 15 min. (Actual observed inlet time from very sharp storm to inlets on Michigan Avenue bridge, 130 ft. wide, was 6 to 7 min.)

For the smaller areas, the formula $Q = ci^{0.75}A$, is used, where c varies from 0.8 to 1.6, the smaller coefficients applying to the more pervious areas. It is suggested that a somewhat more accurate formula would result if the exponent were reduced to 0.70, or even to 0.65.

Cleveland, Ohio.[3]—The rational method is used and a curve made up from local precipitation records. Coefficients of runoff are assumed according to local conditions, varying from 0.60 to 1.00. Inlet time is assumed as 7 to 8 min. Empirical formulas, usually McMath's or Parmley's, are used as a check.

New Orleans, La.[4]—The local situation requires that all drainage water be pumped and financial conditions have made it impossible to provide adequate drainage facilities. The system has, therefore, been constructed and is being extended with the object of providing a uniform retardation rather than the prompt removal of all drainage water. Drain capacities have been fixed on the basis of the Advisory Report of

[1] Arthur W. Kreinheder, Commissioner of Public Works.
[2] C. D. Hill, Engineer, Board of Local Improvements; William R. Matthews, Assistant Engineer in Charge, Bureau of Sewers.
[3] Robert Hoffman, Commissioner and Chief Engineer, Department of Public Service.
[4] George G. Earl, General Superintendent.

1895, using runoff curves based upon the Bürkli-Ziegler formula, with varied coefficients, so that the quantity computed as runoff from 50 acres would vary from 15 to 100 cu. ft. per second. The same method is still followed, since local conditions make it more advantageous to provide equally good drainage over the whole area served than more adequate drainage for limited areas only, with resulting increase in flooding in other areas. The limit of capacity of the present works is 7 in. in 24 hours.

Maximum rainfalls of 5½ in. per hour for 30 min. and ¾ in. per hour for 12 hours are experienced occasionally. Actual coefficients of runoff vary from very low figures up to 1.00 depending on the saturation of the earth. In New Orleans, the direction of travel of storms and the part of the storm in which the heavy downpours occur are factors of great importance in affecting the runoff.

New York City.[1]—Each borough of the city has its own engineering department and makes its own designs, but the principal features must be approved by the Board of Estimate and Apportionment. All of the boroughs now use the rational method in determining runoff. Rainfall intensity is estimated by formulas as follows:

$$\text{In Manhattan borough, } i = 150/(t + 16)$$
$$\text{Brooklyn, The Bronx, and Queens boroughs, } i = 120/(t + 20)$$
$$\text{Richmond borough, } i = 8.91/t^{0.5}$$

The Board of Estimate and Apportionment uses[2]

$$i = \frac{47}{(t + 8)^{0.75}} \text{ for 15-year frequency}$$
$$i = \frac{43}{(t + 8)^{0.75}} \text{ for 10-year frequency}$$

and varies the application according to local conditions. Inlet time is taken as 4 min. for Manhattan, 5 to 10 min., in Brooklyn, 5 in Queens, 10 in The Bronx, and 5 to 7 or more in Richmond. Runoff coefficients generally used are 0.60 for Manhattan, 0.30 to 0.50 for Brooklyn, 0.44 for The Bronx, 0.30 to 0.60 for Queens (0.10 for park areas), and 0.65 for Richmond. The Board of Estimate and Apportionment uses 0.65 for well-built areas, modifying it according to local conditions.

Philadelphia, Pa.[3]—The rational method is used. Rainfall intensity is taken from the formula $i = \frac{116}{t + 17}$, which is estimated to represent a storm occurring, on the average, once a year. Runoff coefficient is taken from the formula $c = \frac{2.8p + 20}{1 + 0.03p}$ where p is the anticipated density of

[1] Kenneth Allen, Sanitary Engineer, Board of Estimate and Apportionment.

[2] See curves in Fig. 84, p. 292.

[3] John A. Vogelson, Chief Engineer, Bureau of Standards.

population in persons per acre. The runoff coefficient used is never less than 0.30. Inlet time is taken as 3 min.

Portland, Ore.[1]—The rational method is used. A rainfall table based on 50 years of record is employed in estimating rate of precipitation. Coefficient of runoff is estimated on basis of local experience, ranging from 0.25 to 1.00. Inlet time is taken as 5 min.

Rochester, N. Y.[2]—The rational method is employed, using the rainfall curve $i = 12/t^{0.6}$. Runoff coefficient is taken as 0.25 for residential districts and 0.90 for fully built-up districts, practically covered by roofs, sidewalks, and pavements. Inlet time is taken as 5 to 10 min., usually 7 min., in residential districts. For very small areas, practically all roofs, the runoff is taken as 4 cu. ft. per second per acre, equivalent to 4 in. per hour rainfall and 100 per cent runoff.

St. Louis, Mo.[3]—The rational method is used. Rainfall is taken from a curve based on local records; no equation for it has been obtained. Coefficients of runoff range from 0.10 to 0.95 (Table 94, p. 291). Inlet time is estimated from observed velocities of flow in gutters with certain arbitrary allowances.

San Francisco, Cal.[4]—The rational method is used. Rate of precipitation is taken from a curve, which indicates a rate of 1.5 in. per hour for 10 min., 0.825 in. per hour for 30 min., and 0.60 in. per hour for 60 min. Runoff coefficients are estimated from assumed future density of population, varying from 0.30 to 0.85. Inlet time is taken at from 3 to 5 min.

Washington, D. C.[5]—The rational method is used. Rainfall has been estimated from the formula $i = 19/t^{0.5}$, but a curve (no formula) has recently been adopted showing higher rates between 35 and 105 min. than are given by the equation, with a maximum excess of 10 per cent at 70 min. This curve represents rates which have been equalled or exceeded three times in 26 years' record. Runoff coefficients range from 0.60 to 0.85. Inlet time is assumed between 5 and 12 min.

FOR HOW SEVERE STORMS SHOULD STORM DRAINS BE DESIGNED?

From an economic point of view, it is possible to compute approximately the point beyond which it is more economical to allow overflowing and to pay or suffer the damages rather than to increase the size of storm-water drains, if it is possible to estimate satisfactorily the damage which may result from flooding and if information is available to indicate the relative frequency of storms of various degrees of severity.

[1] O. Laurgaard, City Engineer.
[2] John F. Skinner, Deputy City Engineer.
[3] W. W. Horner, Chief Engineer, Sewers and Paving.
[4] M. M. O'Shaughnessy, City Engineer.
[5] J. B. Gordon, Sanitary Engineer, District of Columbia.

Practically, however, such computations are of little significance. Local circumstances and conditions, physical and financial, have usually a controlling effect upon the extent to which such drains can be designed to care for extreme maximum rainfalls. The legal responsibility of the community is also an important consideration, although it should not be controlling, since any damage from overflowing must be suffered by members of the community, if not by the entire community as a municipality.

The responsibility of a city for damages of this kind is generally held to depend upon the character of the storm and the courts[1] have held that

> . . . rainfalls are differentiated for judicial purposes into ordinary, extraordinary, and unprecedented classes. Ordinary rain storms are those which frequently occur, extraordinary storms are those which may be reasonably anticipated once in a while, and unprecedented storms are those exceeding any of which a reliable record is extant. The usual rule in determining the responsibility of a city was stated many years ago by the New York Court of Appeals, 32 N. Y. 489, as follows: "If the city provides drains and gutters of sufficient size to carry off in safety the ordinary rainfall, or the ordinary flow of surface water, occasioned by the storms which are liable to occur in this climate and country, it is all the law should require."

The question of what constitutes "ordinary" storms still remains. Are storms which may reasonably be expected to occur on an average once in 10 years ordinary or extraordinary? There seems to be no way of satisfactorily answering this question and it will be necessary for the engineer to decide in each case what is the reasonable condition to be met. The abstracts of legal decisions quoted upon the following pages may aid the judgment, particularly with respect to the legal responsibility of a municipality for flooding due to inadequate storm drains and combined sewers.

Generally, greater damage will result from the flooding of a main or trunk sewer than from that of a branch drain or lateral; yet the damage from overflowing upon a large number of branches may be more serious than would result from flooding of the main sewer. Moreover, it is usually simpler and less expensive to relieve or duplicate a main sewer than to rebuild many small laterals. The additional cost of constructing the latter of ample size when first built will generally be relatively moderate, while the additional cost of a main sewer large enough to care for the most severe storms may be prohibitive, particularly if it is to be designed to meet future conditions, which may not exist for many years to come. It is, therefore, generally advisable to build branch or lateral sewers of the capacity which will ultimately be required, giving the mains and sub-mains a capacity sufficient to provide for the growth for some

[1] *Eng. Record*, 1912; **66**, 617.

years, with the expectation that new relief sewers will ultimately be required to care adequately for the entire runoff from the district.

ABSTRACTS OF LEGAL DECISIONS RELATING TO FLOODING OF SEWERS[1]

Alabama, 1902.—A city for the efficiency of its sewers is bound to make provision for such floods as may be reasonably expected from previous occurrences, though at irregular and wide intervals of time. (*Arndt* vs. *City of Cullman;* 31 So. 478; 132 Ala., 540.)

Delaware, 1888.—In an action for damages to property from an overflow of a sewer during a severe storm caused, as alleged, by the insufficiency of the sewer and an obstruction in it, it is for the jury to determine whether the injury was caused by the insufficiency of the sewer or any obstruction in it owing to the neglect of the city, or by the magnitude of the storm, discharging a greater quantity of water than might reasonably be expected from past experience. (*Harrigan* vs. *City of Wilmington;* 8 Houst., 140; 12 Atl., 779.)

Delaware, 1893.—A city is not liable for damages caused by backwater from a sewer caused by an excessive and phenomenal rainfall against which the city could not reasonably be bound to provide. (*Hession* vs. *City of Wilmington;* 40 A 749.)

Delaware, 1893.—The testimony of an engineer as to the necessary capacity of a sewer in a particular locality for ordinary occasions, is proper evidence of what is an extraordinary rainfall. (*Hession* vs. *City of Wilmington;* 27 Atl., 830.)

Illinois, 1897.—Where a city has provided sewers or drains of ample capacity to carry off all water likely to fall or accumulate upon the streets on all ordinary occasions, it is not guilty of negligence in failure to anticipate and provide for unanticipated and extraordinary storms. (*City of Peoria* vs. *Adams;* 72 Ill., App. 662.)

Illinois, 1901.—A municipal corporation must see to it that the outlet of its sewers is of ample capacity to carry off all the water likely to be in it, but it is not liable for damages caused by an extraordinary and excessive rainfall. (*City of Chicago* vs. *Rustin;* 99 Ill., App. 47.)

Iowa, 1896.—The fact that a city has notice that drains constructed by it to carry off street surface water are insufficient, fails to use ordinary diligence to make such changes as appear reasonably necessary to make the drains serve the purpose intended, does not render the city liable for the resulting overflow of private property where it did not accelerate the flow of the water, or collect the same and discharge it on such property otherwise than it would naturally have been discharged thereon, and it was not negligent either in devising or in adopting the plans of the

[1] Taken from "American Digest, Municipal Corporations."

drains. (*Knostman & Petersen Furniture Co.* vs. *City of Davenport;* 99 Iowa 589.)

Kentucky, 1881.—A municipal corporation is responsible for damages caused by the want of due care and skill in constructing a sewer, and also for the insufficient size or capacity thereof. (*City of Covington* vs. *Glennon;* 2 Ky. Law Rep., 215.)

Massachusetts, 1903.—Where, in an action against a city for damages arising from water coming on plaintiff's premises through a city sewer, there was no evidence that the sewers were defective in construction or obstructed or out of repair and nothing to show that they were established otherwise than by persons acting as public officers under the statute, and the proof tended to show that the defect, if any, in the sewers was in the system which failed to carry off immediately a great accumulation of water due to a heavy rainfall, plaintiff could not recover. (*Manning* vs. *City of Springfield*, 184 Mass., 245.)

Minnesota, 1897.—A city which, in grading a street, constructed an embankment across a stream, making a culvert for the water to pass through, cannot be held liable for damage caused by the insufficiency of such culvert to carry off the water during an unusual storm, unless such insufficiency resulted from a failure to use ordinary care or skill in its construction. It was not required to anticipate such storms as, from the history of the country, would not reasonably be expected to occur, and if it employed competent engineers who were justified in believing and did believe that the culvert was of sufficient size, it was not negligent. (*Taubert* vs. *City of St. Paul, Minn.*; 68 Minn., 519; 71 N. W., 664.)

Missouri, 1894.—Where the negligence of a city in failing to keep its sewers open contributed to the damage to property, it is liable although the rain causing the damage was of an extraordinary character. (*Woods* vs. *City of Kansas;* 58 Mo. App. 272.)

Missouri, 1901.—Where a city set up the defense that the breaking of a sewer was caused by the act of God manifested in an unusual rainfall, and there was no evidence that the sewer was defective by reason of improper construction and failure to repair, and that the rainfall was of an unusual character, it was proper to charge that if an unusual rainfall would have caused the breaking of the sewer notwithstanding its defect then the city was not chargeable with negligence; but if the breaking was caused by such defects or if it was caused by such defects commingled and concurring with unusual rainfall then the city was liable (*Brash* vs. *City of St. Louis;* 161 Mo., 433.)

Missouri, 1903.—Where a rain storm such as had never occurred before, caused a flooding of the lands from a sewer, no greater than would have occurred under natural conditions, the sewer having been scientifically built according to the best judgment of the engineers and

having a sufficient capacity under ordinary conditions, the injury results from an act of God, for which the city is not liable. (*Gulath* vs. *City of St. Louis*, 179 Mo., 38.)

New York, 1861.—There is something very like a contract to be implied from the construction of a sewer at the expense of the adjacent property, that it may be used to drain the property thus charged with its construction, and it would seem that the adjacent property holders have a right to open drains into it; and in a suit by such adjacent property holder who had opened his drain into the sewer upon his own responsibility and whose premises had been flooded by backwater through the drain in a freshet, it was held that a verdict giving him damages must be sustained. (*Barton* vs. *City of Syracuse*, N. Y.; 37 Barb. 292 affirmed (1867); 36 N. Y., 54.)

New York.—A sewer in the city of Troy, built by the owners of land through which it passed and by the city where it passed through its streets and alleys, passed through the premises of plaintiff and emptied into the Hudson river. Another sewer built by the city was connected with it. One T. petitioned the common council for leave to enlarge the opening between the sewers. This was referred to a committee with power. The city commissioner, whose duty it was to look after and inspect sewers, authorized the change to be made. In doing the work T. built a wall across the sewer first mentioned, partially obstructing the outlet, and diminishing the capacity of the sewer, by reason whereof it became clogged and filled up and, a storm occurring, the accumulated water burst open the sewer upon plaintiff's premises, causing damage. T. presented his bill for the work to the common council, which was audited and paid. Held, that the city was chargeable with notice of the obstruction and was liable for the damages resulting therefrom. (*Nims* vs. *City of Troy;* 59 N. Y., 500.)

New York, 1902.—Where a municipality has constructed and maintained a sewer adequate for all ordinary purposes, it is not liable for injuries to abutting owners, caused by overflow of the sewer due to a storm of extraordinary violence. (*Sundheimer* vs. *City of New York;* 79 N. Y. S., 278; 77 App. Div. 53, reversed 1903.)

Pennsylvania, 1882.—In an action against a municipal corporation for damages for injuries sustained by the bursting of a sewer, owing to its negligent construction by defendant, when it appeared that owing to an extraordinary flood the breakage would have happened even if the negligence complained of had not existed, no damages can be recovered. (*Bolster* vs. *City of Pittsburgh;* Leg. J 204.)

Pennsylvania, 1902.—The mere omission of municipal authorities to provide adequate mains to carry off the water which storms and the natural formation of the ground throw on city lots and streets, will not sustain an action by an owner of land, against the municipality, for

damages arising from the accumulation of water. (*Cooper* vs. *City of Scranton;* 21 Pa. Super. Ct. 17.)

Texas, 1894.—Where a lot owner knows that his premises will be flooded in case of a heavy rain, unless a certain city drainpipe in the street adjacent thereto is cleaned out, and gives no notice of it and makes no effort to remedy the defect, he cannot recover of the city, damages caused by flooding his premises during such storm. (*Parker* vs. *City of Laredo;* 9 Tex. Civ. App. 221; 28 S. W., 1048.)

West Virginia, 1896.—A city is not bound to furnish drains or sewers to relieve a lot of its surface water. (*Jordan* vs. *City of Benwood,* W. Va.; 42 W. Va., 312.)

CONCLUSION

While the problem of determining the quantity of storm water to be carried by drains is still difficult and indeterminate, much advance has been made during recent years in the methods of attacking it. This has been due largely to the securing of accurate records of rainfall, showing duration and intensity. More information showing the actual runoff from rains of known intensity upon areas carefully studied to determine their local characteristics (similar to that given in the next chapter), and observations of the time required for the water to reach the sewers, is very much needed, particularly to assist the engineer in making a judicious selection of the coefficient of runoff. There is also need for detailed and long-continued studies of the distribution of rainfall within areas of comparatively small extent, say up to 5 square miles, in order to furnish definite information relating to the area covered by heavy storms, and the rate of diminution of intensity of precipitation as the distance from the center increases. The older empirical formulas have largely given way to rational methods of computation, which enable the engineer to exercise his judgment more readily and design structures peculiarly adapted to local conditions.

CHAPTER X

GAGING STORM-WATER FLOW IN SEWERS

Methods of gaging the flow of water have been referred to in Chapter IV upon Measurement of Flowing Water. As a general rule, weirs, current meters, or other measuring devices are impossible of employment in gaging flood flows and recourse must be had to computation of the discharge from observations of the depth in the sewers, and from the known or measured slope and known or assumed conditions, such as roughness, affecting the flow; although if the depth be observed at a critical section, the slope is not an element in the computation of discharge.

The City of New York in 1926 installed a large Venturi flume in one of the sewers in Manhattan Borough which is shown in Fig. 56, p. 166. It is probable that this will furnish more accurate results than have previously been obtained in storm-water gagings; but in some cases the obstruction to flow caused by such a flume or any other device would not be allowable.

As storms are likely to occur at any time, and observers cannot be constantly on duty, automatic recording gages for showing the depth of sewage at any moment are practically indispensable. At least two of these are necessary in order to determine the slope of the water surface, which is frequently very different from the slope of the sewer. In most gagings which have been made, however, but one instrument has been used, and the hydraulic slope has been assumed equal to the slope of the sewer, though such measurements are likely to be considerably in error. In addition to the recording depth gages, it is desirable to have a number of maximum depth gages, which show the greatest depth of sewage at the point of installation since the last observation, but give no further information.

WATER-STAGE RECORDERS

There are several kinds of automatic recording gages for showing the elevation of the sewage or water level at a gaging point, but all of them belong to one of two general types, float gages and pneumatic pressure gages. Recorders of either type are also applicable to weirs where a continuous record of the head upon the weir is desired, and some of them can be arranged to show on a single chart the elevation of water at two points, as at the entrance and throat of a Venturi flume.

As noted in Chap. VII upon Precipitation, it is extremely important that every automatic gage should have a good clock movement, that it should be regulated to keep correct time and synchronized with the clocks of all other gages the records of which may be studied jointly. If the clocks were furnished with dials, the regulation and synchronizing would be greatly simplified, but few of the gages now on the market have such dials. Electrical operation of the clocks may be practicable on large works where several gages are employed.

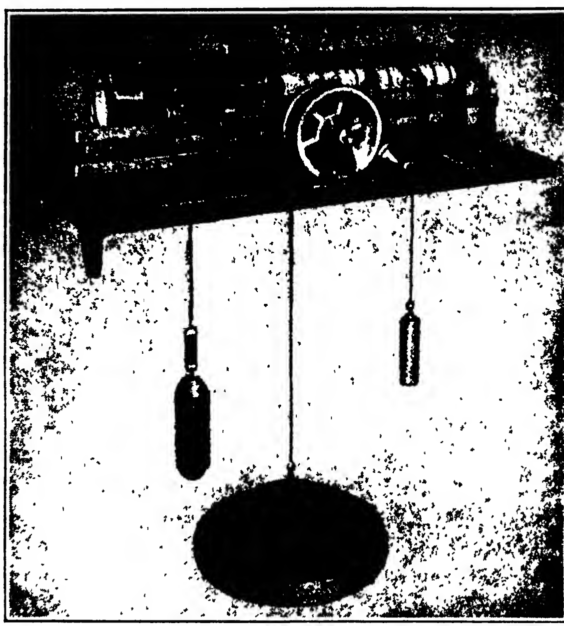


FIG. 95.—Au water-stage recorder.

Float Gages.—In gages of this type, a float contained in a pipe or other suitable guide in which the sewage stands at the same height as in the sewer, is connected with a recording apparatus through the medium of a cord, chain, tape, or by a stiff rod or tube.

The Au water-stage recorder, shown in Fig. 95, is made by Julian P. Friez & Sons, Baltimore, Md. It consists essentially of a float from which the rise and fall of the water surface is transmitted to a float wheel, using two cables and a counterweight; a device operated by the float wheel for reducing and transferring the motion of the float wheel and recording it on the record sheet; and a mechanism for carrying and driving the record sheet. The recorder is covered by a metal case

with glass top. Its dimensions are 11 by 12 by 26 in. and weight 50 lb. The smallest well that will accommodate the float and counterweight is about 18 in. in diameter. Changes in stage of the water surface are transmitted to the record sheet through the float wheel, the shaft of which carries two driving gears each engaging a horizontal movable rack that carries a pencil for making a graph of the rise and fall of the water surface on the record sheet. The effective length of run of each rack is 10 in. which is the width of the record sheet. The first rack carries the pencil which records fluctuations of low water; the second, that for higher stages. If only a small range has to be covered, but one rack is needed. By combining different sizes of float wheels and driving

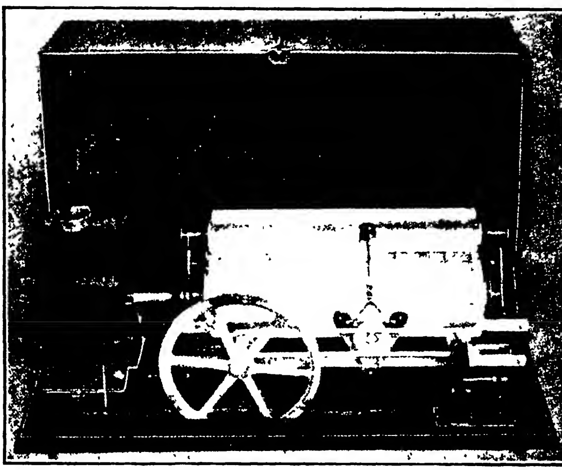


FIG. 96.—Stevens water-stage recorder.

gears, ranges in height of water from 5 to 40 ft. may be accommodated. The recorder is provided in six models for various time scales, ranging from 1.2 in. to 28.8 in. for one day. It should be noted that the ease and accuracy with which a record can be read depends on the relative magnitude of the height and time scales as much as on the individual magnitude of either.

Stevens Water-stage Recorder.—In this instrument (Fig. 96), the pencil is moved horizontally by a belt controlled by a wheel over which a cord from a float passes. The record is made upon a horizontal cylinder around which a sheet of paper is made to revolve, being reeled off from one roll and on to another, so that it is possible to use very long sheets of paper and keep continuous records for long periods of time without renewing the record sheet. A mechanism is also provided by which the motion of the pencil carriage is reversed after reaching the

limit of its motion in one direction, and in this way it is possible to record an unlimited range in elevation without reduction of the scale.

The time scale of the Type 6 recorder, which is especially designed for sewer gagings, is 0.6 in. per hour. Any desired scale of gage heights can be furnished. A scale of 5 in. to the foot may be provided where great accuracy is required, and where the fluctuations do not exceed 4 ft.

The spring-driven clock movement may be replaced by one driven by a weight, in case there is sufficient space available for the movement of

Fig. 97.—Gurley water-stage register.

the weight. An electrically actuated recorder, located at a distance from the float chamber, can also be provided. This instrument is made by Leupold, Voelpel & Co. of Portland, Ore.

Gurley Water-stage Register.[1]—The instrument shown in Fig. 97 is planned to show fluctuations in water level to natural scale, while the time scale may be as large as 8 in. to 24 hours. The pencil is driven horizontally by means of the clock, while the cylinder is rotated as the float rises or falls. By the use of sprocket wheels of other sizes, the depth scale may be varied and smaller time scales are possible by changing the pitch of the driving screw. A weight-driven clock may be substi-

[1] Made by W. & L. E. Gurley, Troy, N. Y.

tuted for that driven by spring and electrical transmitting and recording apparatus can be provided by which this record can be kept at a distance from the float chamber. This gage is equipped with a clock dial to facilitate regulation.

Builders Iron Foundry[1] Water-level Recorder.—In this gage the cord from the float moves an arm carrying a pen in front of a circular chart, which is rotated by clockwork (Fig. 98). The pen accordingly moves in a circular arc and the time scale varies with the position of the pen. The

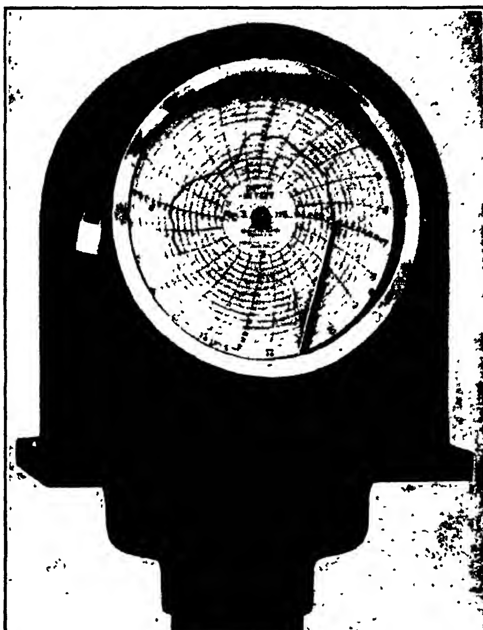


Fig. 98.—Water-level recorder (Builders Iron Foundry).

instrument is enclosed in a cast-iron box mounted upon a hollow standard through which the float cord passes. It is made in two sizes, having 8- and 12-in. dials.

Obviously, with this gage, the scale of heights as recorded upon the chart will depend upon the range to be covered and the size of the chart. A rectangular chart is not necessary for records of this kind, and the only disadvantage of this form of record is that the time scale is unduly small when the pen is in its lowest position.

Builders Iron Foundry, in some cases, also has constructed a modification of the recording instrument of the Venturi meter for use with a

[1] Providence, R. I.

float, to indicate and record directly the rate of flow over a weir, and also to integrate these rates and show on a recorder the total quantity passed.

Pneumatic Pressure Gages.—In these gages a diaphragm box or pressure chamber is immersed in the liquid and the changes in pressure resulting from the rising or falling surface are transmitted through a small pneumatic tube to a recording apparatus located at any convenient point.

Figure 99 shows an instrument of the diaphragm type. It is made for either 8- or 12-in. charts. These instruments are made by The Foxboro Company of Foxboro, Mass., and by The Bristol Company of Waterbury Conn.

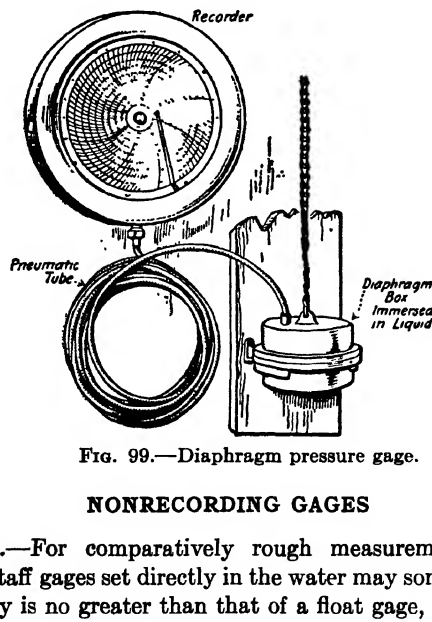


FIG. 99.—Diaphragm pressure gage.

NONRECORDING GAGES

Staff Gage.—For comparatively rough measurements by direct observation, staff gages set directly in the water may sometimes be used. Their accuracy is no greater than that of a float gage, and as their use requires the constant presence of an observer, they are seldom of service in gaging storm flow in sewers.

Point Gage.—Accurate determination of the elevation of the water surface may be made by means of the point gage, one form of which consists of a plumb bob suspended by a fine wire, which passes over a wheel at the end of an arm held horizontally over the water. By a suitable scale marked on the horizontal arm, readings are obtained by lowering the plumb bob until it just touches the water. When the determinations have to be made in dark or inaccessible places, so that the point of the bob cannot be seen, an electrical contact may be brought into use. In this arrangement one pole of a battery is connected to the wire

carrying the plumb bob, while the other pole is connected through a delicate galvanometer to an iron cylinder surrounding the plumb bob and inserted in the water, thus forming a "stilling box." When the plumb bob touches the surface of the water, sufficient current passes to deflect the galvanometer which is placed at some convenient point near the scale board.[1]

It is also possible to make accurate measurements to the water surface from a considerable height, by swinging the bob and noting the measurement when the point just cuts the surface of the water.

Hook Gage.—The most accurate gage is the hook gage, invented by Boyden about 1840. This takes advantage of the surface tension of the water surface and consists, as the name implies, of a hook attached to a rod carrying a scale, which may be moved up and down or clamped to a supporting contrivance provided with a slow-motion arrangement and vernier. The gage is operated by lowering the rod until the point of the hook is below the water surface; the rod is then raised slowly until a protuberance on the water surface is noted just over the point of the hook. The point of the hook does not break the surface of the water immediately, but carries the surface film up with it and the beginning of this phenomenon can be noted accurately by watching the reflected light upon the surface. In a good light, with suitable verniers, differences as small as 0.0002 ft. can be determined.

A gage of this character, known as the Boyden hook gage, is on the market. This gage has a frame of wood 3 ft. long by 4 in. wide, in a rectangular groove in which is made to slide another piece carrying a metallic scale graduated in feet and hundredths from 0 to 2 ft. Connected with the scale is a brass screw passing through a socket fastened to another sliding piece, which can be clamped at any point upon the frame and the scale with hook moved in either direction by the milled nut or slow motion screw. The scale is provided with a vernier which enables the setting to be read to thousandths of a foot. This gage has a number of disadvantages, particularly when used in connection with sewer gagings. The most serious are: (1) The material is largely wood, which is objectionable for permanent installations in damp places. (2) The zero and vernier of the gage are at one end of the instrument, while the slow-motion screw is at the other end, thus making it very awkward in operation.

A much more satisfactory type of hook gage is the Emerson gage, illustrated in Church's "Hydraulic Motors." This instrument is accurate, durable, and convenient to use, but is heavy, not very portable, and decidedly expensive.

Acting on suggestions from Metcalf & Eddy, W. & L. E. Gurley have placed on the market the hook gage shown in Fig. 100. This is con-

[1] *Eng. Rec.*, 1913; **66**, 192.

structed wholly of noncorrosive metal, is light, strong, and has an adjustable hook. A clamp permits its rapid adjustment to any position within the limits of the rod while a slow-motion screw adjacent to the vernier allows the desired accuracy of setting.

The Pacific Flush Tank Co. also makes a simple hook gage of all-metal construction.

Stevens Weight and Hook Gage.—This simple indicating gage (Fig. 101) consists essentially of a reel on which is wound a graduated metal tape. The zero end of the tape is fastened to the reel while the other end

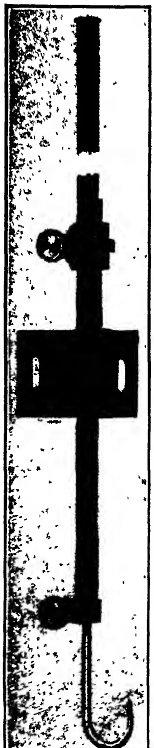


Fig. 100.—Hook gage suggested by authors. (*Gurley.*)

Fig. 101 —Stevens weight and hook gage.

is fastened to the weight hook. Thus, the graduations read downward toward the water.

The hook is shaped like a two-pronged anchor. On the tips of the prongs are two adjustable brass points. These points are always in a level line, the weight being symmetrical and well balanced.

The index is conveniently fixed to the reel base and always points to the correct gage height when the tips of the hooks are at the level of the water surface.

Where errors might result from lowering the "anchor" into the water, as in a stream with high velocity, the instrument may be used as a point

or weight gage, noting when the point on the bottom of the weight first cuts the surface of the water.

Maximum-flow Gages.—In its simplest form, a maximum-flow gage consists of a strip of wood held securely in a sewer in a vertical position and so coated that it will show at a glance how much of the strip has been submerged at any time since it was put in place.

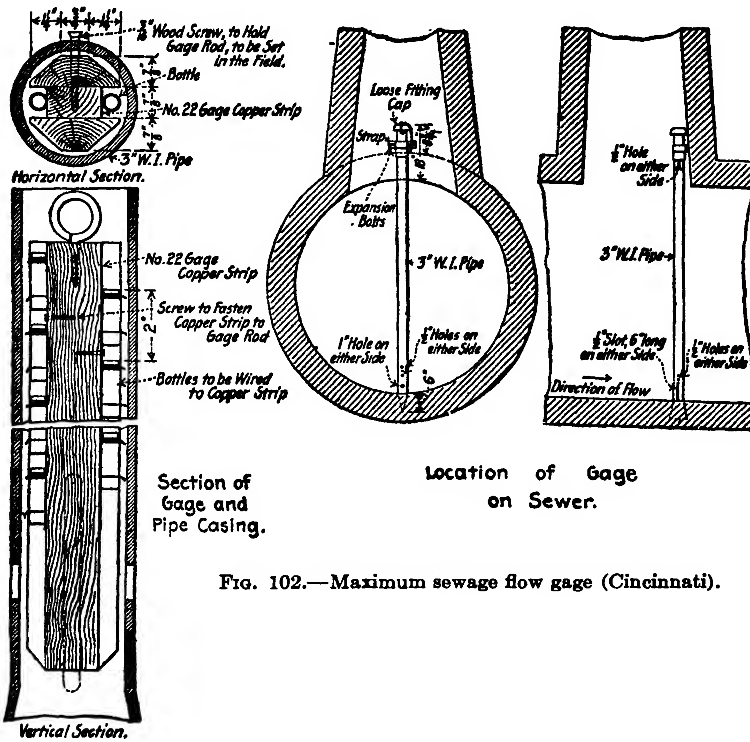


FIG. 102.—Maximum sewage flow gage (Cincinnati).

The first maximum gages consisted simply of strips of wood coated with whitewash and fastened firmly into the manholes. Later, to make more certain the determination of the point to which the sewage had risen, sand was imbedded in the whitewash and mucilage was used to help retain the sand, it being supposed that the sand would fall away from the portions which had been wet.

Such gages have not proved generally satisfactory, owing principally to the effect of the moisture in the sewer air, which affects the coating, so that it gradually disappears, or else it absorbs so much moisture that the water line cannot be clearly distinguished.

Other washes have been tried, in an attempt to obtain a coating which could be used satisfactorily, but the degree of success attained is not high. The authors have found that a wash composed of soap, corn starch, and methylene blue applied to the brickwork of a large sewer, is satisfactory as indicating the height reached by the water, if the coating is fairly fresh. After a week or so, however, the coating deteriorates so that it is no longer possible to distinguish the water line with certainty.

The illustration (Fig. 102) shows a maximum-flow gage which was devised by the sewerage engineers of the City of Cincinnati (H. S. Morse, Engineer in Charge) to overcome these defects. It will be noticed that to clamp the rod firmly in place it is put inside a 3-in. steel pipe secured in an upright position, with openings near the bottom to allow the sewage in the pipe to rise and fall with that in the sewer. The rod itself is supported by a wood screw held by a "bayonet joint," a slot in the pipe having a right-angled change of direction. The rod itself carries a series of vials so fastened that their mouths are 1 in. apart vertically. It is therefore obvious that the sewage has been at least as high as the highest vial which is found filled; and it is only necessary to invert the rod, empty the bottles and then replace the rod, to have the gage ready for another observation. This gage has proved satisfactory under ordinary conditions, but the results were not satisfactory when velocities greater than 8 ft. per second were encountered. A similar gage has been used at Pawtucket, R. I., by George A. Carpenter, City Engineer.

In Boston, Mass., maximum sewer gagings are recorded by a circular box float, with a central opening through which a vertical guide rod runs. The fit is a very loose one, but a pair of sheet brass springs attached to the float press lightly on the guide and hold the float at the highest elevation to which it is lifted.

SETTING SEWER GAGES

Local conditions will determine the locations of the points at which gages should be established. It is, however, always necessary that the sewer for a considerable distance upstream and for a less distance below the gage should be in such condition that the quantity flowing can be computed from the depth in the sewer. If the computation of flow is to be made by Kutter's or a similar formula, the cross-section and slope must be uniform, there must be no curves, no inlets or obstructions to cause disturbance in the flow, and the condition of the interior should be known so that a coefficient of roughness can be applied with a good degree of accuracy. Moreover, the velocity of flow in the sewer should not be great. In order to be sure of the results, it is necessary to have gages at each end of such a stretch of sewer, to determine the slope of the water surface.

If it is possible to locate a gage at or a short distance above a section which will be a critical section[1] under all possible conditions of flow, this will generally be the most advantageous thing to do. At a critical section, the discharge is a direct function of the depth of flow, and uncertainties may be eliminated in large measure if gagings are made in this way.

In laying out a new sewer, it will generally be possible to locate a critical section at any desired point, by flattening the slope above and steepening it for a short distance below the location selected. The steeper slope must be steep enough to constitute a chute. Under some conditions a drop manhole may be used instead of a chute. When gagings are to be made in an existing sewer, the problem is more difficult. Sometimes it may be possible to select a point where the slope changes, but where the inclination of the lower section is not steep enough to constitute a chute, and artificially provide a critical section by the construction of a low dam in the sewer. In such a case the effect of backwater resulting from the dam must be considered, and it will also be necessary to make sure that the section will remain a critical section at high flows as well as at low ones.

Gaging apparatus should be installed in a separate chamber or gaging manhole at one side of the sewer, to protect the instruments and make them easily accessible for observation or adjustment. This chamber should be connected with the sewer so that water will stand in the chamber at the level of the sewage in the sewer. It is desirable to have a small flow of clean water into and through the gaging chamber and thence to the sewer, in order that the liquid surrounding the instruments or floats shall be water and not sewage, thus avoiding clogging and derangement, as well as rendering the chamber a much pleasanter place to enter than it would be if partly filled with stagnant sewage.

An excellent example of a special gaging chamber or manhole is shown in Fig. 103, an illustration of the chamber constructed by the sewer division of Cincinnati. This is arranged for a gage of the diaphragm type, from which the pressure pipe will be conducted through a wrought-iron pipe to a recorder in an iron box mounted at the curb or in a house. If a float gage were to be used, it would be necessary to locate the recorder within the chamber, upon a post, or in a building directly over it. The location within the chamber is objectionable, owing to the rapid corrosion of the clockwork and other parts of the instrument, as well as to the effects of moisture upon the paper chart. Where the sewer is in a street, it is usually not possible to locate a building or even an iron post and box directly over the float chamber, unless the latter is extended so as to lie at least partly under the sidewalk. While the design of the gaging chamber is good, the connection between it and the sewer may

[1] See p. 102.

possibly be criticized, because it is not normal to the inner surface of the sewer, as a true piezometric connection should be. Any probable error

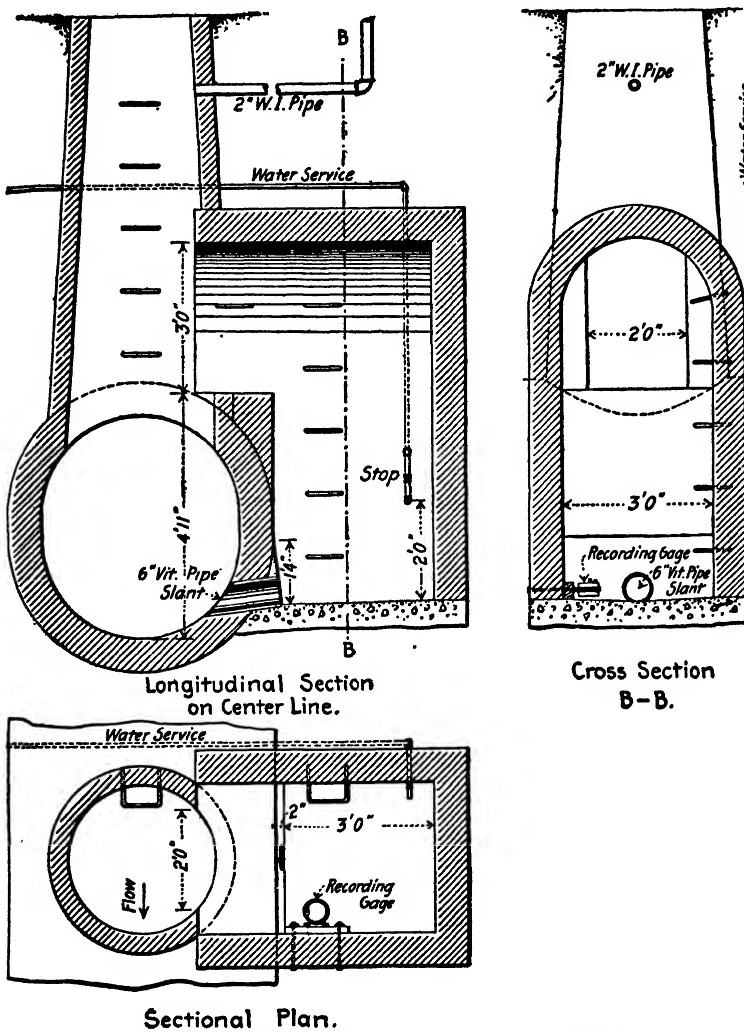


FIG. 103.—Gaging chamber, Cincinnati sewerage system.

from this source would be less than the limits of accuracy of measurements with gages of this type.

ACTUAL MEASUREMENTS OF STORM-WATER FLOW

It must be admitted that the determination of the runoff factor from actual gagings is extremely unsatisfactory. Only a limited number of such gagings have been made and even the best of these leave much to be desired; the coefficients deduced from them can be considered only as approximations. Nevertheless, these measurements are of much importance, not only because they furnish the only experimental determinations of the runoff factor which are to be had, but because a careful study of them aids materially in training the judgment and in arriving at a clear and full conception of the problem.

In the present state of our knowledge, only sound judgment based upon experience and clear thinking, with a full conception of the various parts of the problem, can be relied upon for the selection of factors to use in the design of storm-water conduits, because the existing gagings are lacking in the determination of important elements and the characteristics of districts are constantly changing with the growth of cities, so that a coefficient which might be applicable today will be totally insufficient for conditions likely to exist in the near future.

For an exact analysis of the relation between precipitation and runoff, it is necessary to know the true rainfall upon the district drained, including the distribution of rainfall over the entire area at all times during the storm, and the true storm runoff, including not only the quantity flowing past the gaging point at all times during and immediately before and after the storm, but the amount which could have been concentrated at this point if the conditions had been favorable. For instance, if the critical precipitation comes at the beginning of a storm when the flow in the sewers is small and the velocity of flow slight, a very considerable portion of the runoff from the surface will be required to fill the sewers. In this case, the velocity of flow will be small, the time of concentration will be long, and the actual maximum rate of flow in the sewer will be materially less than the real rate of runoff. If, on the other hand, the critical precipitation occurs after a long period of moderately heavy rain, particularly if accompanied by melting snow, when the storage space in the sewers is largely filled and the velocity of flow is at a maximum, the quantity actually flowing in the sewer will represent very nearly the true runoff from the storm and the time of concentration will be a minimum. Whether or not the storm-water inlets are adequate to admit water into the sewer as rapidly as it reaches them is also of importance.

Rainfall.—The true rate of precipitation upon the district gaged must be known for each instant during the storm. If the district is a small one, a single recording rain gage near the center of the area may be sufficient; otherwise several such gages will be required, distributed over the area, since the intensity of rainfall frequently varies widely in comparatively short distances. It is extremely important that the gage clocks

be carefully adjusted to keep correct time and to agree with one another and with those of the sewer gages, otherwise the deductions drawn from the records of several gages, and also those relating to time of concentration as shown by a comparison between rain and sewer gages, may be materially in error.

The matter of travel of the storm is also of importance. It is evident that if downpour begins at the most distant point of the drainage area and travels toward the outlet, the resulting maximum flow in the sewer will have progressed some distance before the portions of the district nearest the outlet begin to contribute water. The result is a decreased time of concentration for such storms and an increased runoff as compared with a storm of uniform intensity over the entire district. Storms in which the travel is in the reverse direction would have the opposite effect.

It is obvious that travel of storms can only be determined by a number of gages suitably located and with the clocks carefully regulated. So far as is known, no records which throw light upon this subject are to be had.

It is evident that where the rainfall record is that of a single gage, particularly if at a distance from the sewer district gaged, the inferences relating to time of concentration, area tributary at time of maximum discharge, and runoff factor may be considerably in error.

Measurement of Runoff.—In the great majority of cases, the estimation of flow has been accomplished by computing the quantity flowing in the sewer, by Kutter's formula, using an assumed value of the coefficient of roughness, and assuming the slope of water surface parallel to the invert of the sewer, an automatic gage being used to record the depth of flow. In many cases, the resulting estimated runoff may be far from the truth. Horner[1] has found that:

> There are marked differences between the grade of the sewer and the water-surface grade. For example, in a 9-ft. sewer for one rain a depth of flow at one point of 4½ ft. was observed; 1,000 ft. downstream the depth was less than 4 ft., though several tributaries entered between, while 500 ft. further downstream the depth was over 5 ft. . . . The sewer is uniform as to grade, size, and condition. The most reasonable explanation of these gagings is that the flow at the upper and lower gages is disturbed, in the case of the upper gage, by a curve 200 ft. upstream, and of the lower by a 3-ft. lateral discharging into the main sewer nearly at right angles 100 ft. above the gage.

The fact that storage in the sewers may result in a rate of flow much less than the rate of storm-water runoff has already been referred to.

Extent of Drainage Area Tributary.—Some of the minor flood flows gaged may have been derived from areas less than the whole drainage

[1] *Jour. West. Soc. Eng.*, 1913; **18**, 703.

area at the particular gaging point. Uncertainty as to the extent of the drainage area actually contributing to the flow, corresponding time of concentration, and rate of rainfall producing the flow, leads to doubt as to the correctness of the runoff coefficients derived.

Characteristics of Sewer District Gaged.—Finally, bearing in mind possible inaccuracies in the determination of the coefficient of runoff, it it necessary to know accurately the characteristics of the district in order to form an opinion of the applicability of the coefficients to other districts. These characteristics are of three classes, permanent, semipermanent, and temporary.

The principal characteristics which may be classed as permanent are the size and shape of the district, the surface slopes, and the character of the soil. Even these are not absolutely permanent, as they are all subject to alteration if extensive grading operations should be undertaken.

Semipermanent characteristics, those which change but slowly, are the extent and kind of the impervious or nearly impervious surfaces, such as roofs and pavements, the extent to which the district is sewered, and the sizes and grades of the sewers. The last items are particularly important in their relation to velocity of flow and to storage in the sewers.

Temporary characteristics relate to conditions existing at the time of gaging, which may be modified radically within a period of a few hours. The most important are those relating to the condition of the ground and roofs, whether and to what extent they are wet or dry, frozen, or covered with snow or ice. Other conditions of minor importance are temperature and wind.

Inlet Time.—A matter of considerable importance in this connection, and one about which definite information is very incomplete and unsatistory, is the "inlet time," or time required for the water falling upon the surface to reach the inlets or catch basins. In many cases, particularly for the smaller districts, the inlet time may constitute a large percentage of the total time of concentration. So important is this matter that the Sewer Department of St. Louis started a special investigation in 1913 to determine the time of inlet and the quantity of runoff for individual drainage districts.

Two special cases have been taken for the work, one an area consisting principally of backyards and alleys in which the slope is slight; the other a whole block closely built and having steep grades. The inlets have been reconstructed, and a chamber containing a V-notch weir built under the street between inlet and sewer. Bristol gages are installed to measure head on the weirs. All adjacent inlets have been enlarged and arranged so that no water can cross over from one inlet area to another, even in the heaviest rains; the exact extent and character of the inlet areas have been plotted.

The values of the runoff factor from these small areas will be of great service in analyzing the results from gages in the sewers.[1]

Records of Measurements.—Having called attention to the defects to which all the recorded storm-water gagings are subject in greater or less degree, it may be well to reiterate that, notwithstanding their imperfections, these gagings are still of importance and should be studied carefully. It is much to be desired that the number of gagings should be increased, particularly for small districts, and that uncertainties and unsatisfactory conditions such as those attending the earlier measurements, be eliminated as far as possible. The work now (1928) in progress in several locations, especially in St. Louis and New York City, seems to offer promise of more extended and more precise information in the future. The first step in the acquisition of complete and accurate data must always be the recognition and avoidance of all possible sources of error and uncertainty.

W. W. Horner,[2] who is in charge of the work at St. Louis, has stated:

In the matter of percentages of runoff, or of rates of runoff for particular intensities, the writer feels that little progress has been made. Engineers are still dealing almost entirely with arbitrary values, or, in a few instances, with values based on personal experiences. Published gagings on existing sewer districts are few, and many of the records are faulty.

Even where the records are unquestionable, the results for different storms vary so widely that engineers have not yet been able to make them conform to a definite theory. The writer has no doubt that, as such records multiply, accompanied with all the information which may be of interest as affecting runoff, some definite values will be developed.

The studies of the first 3 years of gagings in St. Louis were rather discouraging and the matter was laid aside for the accumulation of further data. With records for 10 years now available, it is again hoped that some law of variation in runoff will become discernible. These gagings, like most of those heretofore recorded, deal with the runoff of mixed urban areas of from 20 to 500 acres in extent. The writer has long felt that the analysis of these records would be much simplified by a study of the runoff of small plots of uniform character. In an effort to secure such data, the runoff from single blocks is being studied in St. Louis, and it is also proposed, in the near future, to make a study of the runoff on large roofs. To complete this series, observations will be made of the runoff from small plots of natural soil.

It is particularly to be borne in mind that in the determination of the runoff factor it is necessary in most cases to assume the time of concentration and take the rate of precipitation corresponding to this time for comparison with the maximum rate of runoff. Material errors in the coefficient may result from erroneous estimation of the time of concentration, which is not constant for a given sewer district.

[1] HORNER, *Jour. West. Soc. Eng.*, 1913; **18**, 703.

[2] *Trans. Am. Soc. C. E.*, 1922; **85**, 133.

Coefficients greater than unity may occasionally result from combinations of frozen ground, melting snow and the like, but are more likely to indicate that the time of concentration has been overestimated.

Progress reports on gagings of storm runoff in a section of Brooklyn, N. Y., containing about 200 acres of which 18 per cent is pervious, 20 per cent semi-pervious, 52 per cent impervious and 10 per cent undrained, show the ratio of runoff to rainfall during 24 storms in 1925. Some of these storms lasted for periods less than the time of concentration, yet in a few of them the coefficients of runoff were as high as in longer storms. The general range of coefficients was approximately from 0.35 to 0.62, with an apparent normal coefficient of about 0.50.[1]

Table 109 contains the most important data relating to all gagings of storm-water flow in sewers which have come to the attention of the authors. Additional data are, in most cases, given in the original publications, to which reference may be had for further details. The wide fluctuations in the resulting value of c for each sewer on which a number of gagings were made probably result partly from using the minimum (computed) value of the time of concentration, instead of the actual time for the particular storm; partly from using storms which did not last long enough for the entire area to contribute to the flow; and partly from causes not known.

Runoff from Small Plots.—A long series of experiments upon the runoff from natural earth surfaces was made for the Miami Conservancy District in the years 1915 to 1919, by Ivan E. Houk, under the direction of A. E. Morgan, Chief Engineer. The actual surface runoff of plots 5 ft. square was collected in tight cans, the plots being surrounded by sheet-metal barriers.

Two series of experiments were made. The first, called the Moraine Park experiments, were made on four plots about 5 miles south of Dayton. Two of them were nearly level, the other two on a hillside; one of each pair was covered with sod, the other being stripped. The surface soil consisted of a yellow sandy loam containing some sand and gravel. A standard rain gage was located near each pair of plots and the precipitation and amount of water collected in the cans was measured after each storm. The comparison is therefore between total rainfall and total runoff and not between rates of rainfall and runoff.

The other series of experiments were made in 1920 on the same plots and on four similar plots near the Taylorsville Dam, two in the bottom of the valley, where the soil is a rich black alluvial deposit underlaid by glacial till, and the other two on the top of the hill near the end of the dam, where the soil is a compact yellow-clay till. Some vegetation was present on one plot in the valley and on both plots on the hill. The effect of various rates of rainfall was approximated by applying definite

[1] *Public Works*, 1927; **58**, 429.

TABLE 109.—VALUES OF COEFFICIENT OF RUNOFF DETERMINED FROM MEASUREMENTS OF STORM-WATER FLOW IN SEWERS AT VARIOUS PLACES

Sewer district	Area, acres	Estimated coefficient of imperviousness	Estimated time of concentration, minutes	Year of measurements	Number of measurements	Values of c			References
						Maximum	Minimum	Mean	
Birmingham, Eng.: (D. E. Lloyd-Davies)									
Mosely St.	312.5	1.00	18	1904	28	1.24	0.34	0.83	4, 6, 11
Charlotte Rd.	232	0.18	12	1904	21	0.29	0.15	0.22	
Bordesley St.	19.32	1.00	6.5	1904	8	1.26	0.93	1.05	
Cambridge, Mass.: (L. M. Hastings)									
Shepard St.	56.5	0.36	20	1900	15	0.86	0.15	0.43	4, 5, 7, 11
Sherman St.	68	0.28	20	1900	7	1.00	0.29	0.65	
Bath St.	223	....	30±	1910–1911	4	0.20	0.14	0.16	
Oxford St.	400	0.29	40	1909	12	0.21	0.09	0.17	
Chicago, Ill.: (S. A. Greeley)									
Winnetka	387	0.10	60	1912	7	0.23	0.15	0.20	4, 5, 11
Evanston	420	0.20	40	1912	5	0.22	0.07	0.14	
Diversey Blvd.	725	0.22	75	1912	2	0.67	0.23	0.45	
Robey St.	2,516	0.08	420	1912	7	0.27	0.07	0.15	
Hartford, Conn.: (F. L. Ford)									
Franklin Av.	477	....	25	1900	3	0.18	0.11	0.16	4, 11, 12
Franklin Av.	263.5	....	22	1901–1902	3	1.30	0.14	0.57	

Louisville, Ky.:									
(J. B. F. Breed)									
Western Outfall	2,500	0 36	80	1910	1	. .	. .	0.32	4, 11
Milwaukee, Wis.:									
(Logemann and Nommensen)	1,138		44	1898	4	0.38	0.17	0.25	4, 8, 11
Newton, Mass.:									
(E. H. Rogers)									
Hyde Brook	350	0 28	20	1907–1908	2	0.43	0 28	0.36	4, 11
New York, New York:									
(Rudolph Hering; Com. of Soc. Munic. Engrs.)									
Sixth Av., Manhattan	221	0 90	20	1887–1888	11	0.84	0.34	0.52	3, 4, 11
Tinton Av., Bronx	56.3	0 68	.	1911–1912	35	0.62	0.27	0.39	13
Cedar St., Richmond	76.8	0.32	16	1912–1913	4	0 40	0 20	0.31	14
Broad St., Richmond	900	0 21	24	1912–1913	3	0 25	0 16	0.21	14
Pawtucket, R. I.:									
(G. A. Carpenter)									
Newell Av.	106	0 35	24	1907–1910	17	0 87	0 26	0 44	4, 11
Philadelphia, Pa.:									
(Bureau of Surveys)									
Twelfth St.	1,306	.	40 ±	1903–1912	32	1 45	0.30	0.79	4, 10, 11
Rochester, N. Y.:									
(Emil Kuichling)									
I	356.9	0 15	44	1887–1888	17	0.16	0.04	0.09	
IV	128 7	0 26	26	1887–1888	16	0.29	0.09	0.18	
IX	133	0.38	23	1887–1888	15	0.42	0.12	0.25	1, 4, 11
X	25.1	0 50	16	1887–1888	12	0.65	0.18	0.38	
XVII	92.3	0.30	24	1887–1888	16	0.37	0.09	0.19	
Seattle, Wash.:									
(H. D. Silliman)									
First Av.	156.4	0 85	10[1]	1914	4	0.55	0.23	0.38	9

TABLE 109.—VALUES OF COEFFICIENT OF RUNOFF DETERMINED FROM MEASUREMENTS OF STORM-WATER FLOW IN SEWERS AT VARIOUS PLACES.—(*Continued*)

Sewer district	Area, acres	Estimated coefficient of imperviousness	Estimated time of concentration, minutes	Year of measurements	Number of measurements	Values of c			References
						Maximum	Minimum	Mean	
Washington, D. C.: (R. L. Hoxie) New York Av.	436	0.60	25	1881 1884–1885	3	1.00	0.48	0.74	2, 4, 11
Wilmington, Del.: (A. J. Taylor) Shipley Run	174	0 65	20	1908	1	. . .	. . . .	0.79	4, 11

[1] Observed: time of flow computed as 20 min.

References:

1. *Trans. Am. Soc., C. E.*, 1889; **20**, 1.
2. *Trans. Am. Soc., C. E.*, 1895; **25**, 81.
3. *Trans. Am. Soc., C. E.*, 1907; **58**, 464.
4. *Jour. Boston Soc. C. E.*, 1914; **1**, 291.
5. *Jour. West. Soc. Eng.*, 1913, **18**, 663.
6. *Proc. Inst. C. E.*, **161**, 5.
7. *Report* of John R. Freeman on Charles River Dam.
8. *Eng. News*, 1901; **45**, 406.
9. *Eng. News*, 1915; **74**, 832.
10. *Ann. Rep. Bur. Surveys.*
11. METCALF & EDDY. "American Sewerage Practice," vol. i, First Edition, 1914; p. 316 *et seq.*
12. *Trans. Conn. Soc. C. E.*, 1900–1901; 133.
13. *Proc. Munic. Eng.*, N. Y., 1913; 373.
14. *Munic. Eng. Jour.* (N. Y.), 1922; 22.

quantities of water in fixed times by means of ordinary garden watering pots.

The most important results of these experiments are given in Tables 110 and 111 below.

Table 110 contains the percentage of rainfall collected in all the Moraine Park experiments in which the depth collected from any one of the four plots exceeded 1 in. It shows that the effect of slope was comparatively slight, while the effect of surface cover was very marked.

TABLE 110.—EFFECT OF SLOPE AND SURFACE COVER ON RUNOFF IN MORAINE PARK EXPERIMENTS

Date	Rainfall, inches	Runoff in per cent of rainfall			
		Sod cover		Bare soil	
		Level	Sloping	Level	Sloping
Aug. 18, 1915	4 22	14	32	38	41
June 3, 1916	1 99	0	2	60	56
Aug. 7, 1916	3.63	0	7	43	27
Sept. 6, 1916	4 32	0	1	44	38
Jan. 8, 1917	2 04	0	0	51	20
June 11, 1917	2 77	0	0	39	26
June 30, 1917	2 90	1	1	60	57
Aug. 25, 1917	3 04	1	0	45	50
Feb. 11, 1918	5 26	44	52	58	49
Feb. 26, 1918	2 54	0	6	58	15
July 23, 1918	3 11	1	4	71	71
Sept. 26, 1918	3.54	0	0	33	35
Aug. 25, 1919	2.56	0	13	67	68
Average		5	11	51	43

The most important conclusions drawn from the Moraine Park experiments, as far as they relate to surface runoff are—

That for extremely small areas, such as the Moraine Park plats, the occurrence and amount of runoff are affected much less by surface slope than by surface cover.

That appreciable surface runoff frequently occurs during intense summer storms when the upper 6 in. of soil are not nearly saturated.

That surface runoff does not occur during some less intense storms, even though the ground is saturated.

That water can be absorbed by the bare soil at times when the soil is unusually dry, at a rate as great as 1.00 in. per hour for intervals as long as 30 min.

TABLE 111.—RUNOFF COEFFICIENTS IN SPRINKLING EXPERIMENTS OF MIAMI CONSERVANCY DISTRICT ON 5-FT. SQUARE PLOTS AT MORAINE PARK AND TAYLORSVILLE

Experiment number	Run number	Length of run, hours	Rate of rainfall, inches per hour	Rate of runoff, inches per hour	Runoff coefficient	Time before runoff began, minutes	Condition of soil at beginning of experiment
Level Bare Soil Plot at Moraine Park							
1	1	0.84	4.06	1.86	0.46	2	Dry and loose
	2	0.90	2.10	1.11	0.53	3	
	3	0.44	1.12	0.46	0.41	2	
	4	0.21	0.81	0.44	0.54	4	
	5	0.86	0.77	0.36	0.47	5	
	6	0.68	0.27	0.04	0.15	30	
	7	0.31	0.18	0.00	0.00	..	
	8	0.26	3.11	1.75	0 56	1	
3	9	1.10	3.26	2.56	0.79	6	Trampled and packed hard; dry
	10	0.96	3.33	2.98	0 90	1	
	11	0.91	3.54	3.16	0 90	2	
	12	1.76	1.70	1.57	0.92	10	
	13	1.35	0.89	0.77	0.82	5	
	14	0.74	0.54	0.48	0 89		
	15	0.76	0.26	0.17	0.66		
Sloping Bare Soil Plot at Moraine Park							
4	16	1.25	3 20	2 30	0.72	4	Dry and loose
	17	1.05	3 41	2.69	0.79	1	
	18	1.00	3.41	2.79	0.82	1	
	19	2.18	1.84	1 38	0.75	2	
	20	1.32	0.91	0.58	0.64	2	
	21	0.72	0.55	0 27	0.49	0	
	22	0.73	0.28	0.05	0.18	5	
Level Sod-covered Plot at Moraine Park							
2	23	0.49	4.17	0.00	0 00	..	Dry and very loose
	24	0.43	14.4	0.4	0.03	42	
	25	0.17	20.4	7.4	0.36	2	
Level Sod-covered Plot in Valley at Taylorsville							
5	1	1.28	3.12	1 99	0.64	3	Dry and hard
	2	0.94	3.18	2.75	0.86	2	
	3	0.99	3.02	2.69	0.89	1	
	4	1.78	1.69	1.19	0.71	4	
	5	1.36	0.88	0.56	0.64	5	
	6	0.77	0.52	0.21	0.40	0	
	7	0.72	0.28	0.02	0.07	0	
9	8	1.74	3.46	2.39	0.69	7	Unusually dry and hard; some cracks
	9	0.93	3.23	2.68	0.83	3	
	10	0.88	3.40	2.93	0.86	1	

TABLE 111.—RUNOFF COEFFICIENTS IN SPRINKLING EXPERIMENTS OF MIAMI CONSERVANCY DISTRICT ON 5-FT. SQUARE PLOTS AT MORAINE PARK AND TAYLORSVILLE.—(*Continued*)

Experiment number	Run number	Length of run, hours	Rate of rainfall, inches per hour	Rate of runoff, inches per hour	Runoff coefficient	Time before runoff began, minutes	Condition of soil at beginning of experiment
Level Bare Soil Plot in Valley at Taylorsville							
6	11	1.88	3.73	1.49	0.40	22	Dry and loose, spaded and raked
	12	1.38	3.63	2.19	0.60	2	
	13	2.14	1.87	1.01	0.54	6	
	14	1.15	1.04	0.51	0.49	3	
	15	0.69	0.58	0.19	0.33	5	
	16	0.57	0.41	0.06	0.15		
Level Weed-covered Plot on Hill at Taylorsville							
7	17	0.92	3.49	3.10	0.89	3	Dry and hard
	18	0.99	3.43	3.22	0.94	2	
	19	1.01	3.36	3.19	0.95	1	
	20	2.11	1.90	1.61	0.85	3	
	21	1.19	1.01	0.80	0.80	3	
	22	0.69	0.58	0.42	0.72	0	
	23	0.65	0.31	0.14	0.45	0	
11	24	1.31	1.53	0.58	0.38	18	Unusually dry and hard; some cracks
	25	1.32	1.51	0.84	0.56	3	
	26	1.28	1.56	1.02	0.65	0	
Sloping Weed-covered Plot on Hill at Taylorsville							
8	27	1.08	3.32	2.77	0.83	3	Dry and hard
	28	0.94	3.40	3.03	0.89	1	
	29	0.95	3.38	3.04	0.90	1	
	30	1.91	1.90	1.49	0.79	3	
	31	1.23	0.97	0.72	0.74	2	
	32	0.62	0.64	0.42	0.66	0	
	33	0.49	0.41	0.20	0.49	0	
10	34	1.33	3.60	2.61	0.73	5	Unusually dry and hard; some cracks
	35	0.96	3.73	3.03	0.81	2	
	36	0.93	3.89	3.42	0.88	1	

That water cannot be absorbed by the bare soil at any time, no matter how dry it is, at a rate as great as 3.00 in. per hour for periods as long as 5 min.

Table 111 contains the principal data relating to runoff, as shown by the sprinkling experiments. They are believed to be especially significant as showing the runoff from saturated soils. Although the soil was comparatively dry at the beginning of each experiment, it soon became

saturated to such a depth that runoff began. By the end of the first run, the soil was saturated to such an extent that the rate of runoff caused by a given rate of precipitation was practically constant. The rates of precipitation were very high. It is to be noted that a reduction in the rate of precipitation in the last runs of an experiment was accompanied by a reduction in the coefficient of runoff.

The time necessary to produce a runoff at the lowest corner of the plot is also of significance, particularly in view of the small size of the plots.

CHAPTER XI

SEWER PIPE

Sizes.—Sewer pipes are manufactured and sold as a commercial product in sizes from 4 to 42[1] in. in diameter.

Certain kinds of pipes, such as cast iron, steel, and reinforced concrete, are either regularly manufactured or can readily be obtained in still larger sizes. For the purposes of this discussion, however, sewer pipe will be considered as limited by the standard sizes of pipes especially made for sewers and not larger than 42 in.

The standard sizes approved by the American Society for Testing Materials (A.S.T.M.) range from 4 to 12 in. by increments of 2 in. and from 12 to 42 in. by increments of 3 in.

Requisites.—The principal requirements of any pipes to be used for sewers are strength, durability, imperviousness, smoothness, hardness, uniformity of size and shape, tightness of joints, and economy of construction.

Obviously, the pipe must have sufficient strength to stand the stresses to which it will be subjected, otherwise it will crack or possibly collapse, and if the sewer is not entirely closed, water will leak in or sewage will leak out, and much trouble and expense is liable to result. It must not be subject to disintegration through the effects of weather, moisture, or acids or alkalies in the soil or in the sewage. It must be sufficiently impervious to prevent the admission of a material quantity of ground water, or the escape of sewage into the earth. It must be smooth on its interior surface, in order to avoid resistances to flow and accumulations of deposits. It must be hard so as to resist erosion. It must be uniform in size and shape in order to avoid projections and irregularities at the joints. The joints must be so designed and constructed that they will be and will remain tight. And finally, the cost of the pipe chosen must be such that it will be economical to use in comparison with other kinds of pipe or, for large sizes, with other types of construction.

Kinds of Pipe.—Two kinds of sewer pipe are made commercially—vitrified clay and cement-concrete sewer pipe.

Drain tile is also made of both of these materials, and differs from sewer pipe principally in that it is made without socket or spigot (the ends of the lengths being merely butted together), it is not usually salt

[1] Sizes larger than 36 in. in vitrified clay pipe, and 24 in. in plain cement-concrete pipe, cannot readily be obtained at the present time (1927).

glazed, and the requirements for material and workmanship are somewhat less severe than for sewer pipe. In the clay tile, this usually means that they are burned at a lower temperature than sewer pipe, which may result in less blistering, smoother interior surface and less distortion from heat. While drain tile is not suited for sewerage work (except in some cases for underdrains), it is important to know something of its characteristics, since, occasionally, significant data upon drain tile are obtainable which may be applicable, with some allowances, to sewer pipes.[1]

Other kinds of pipe are sometimes used for sewers, notably reinforced concrete, which has been employed to some extent in sizes for which monolithic concrete or brick sewers would also be considered. Under special circumstances, cast-iron, steel and even wood-stave pipes are also employed.

VITRIFIED CLAY PIPE

Material.—Clay pipe is manufactured from surface clay, fire clay, or shale, or a combination of these materials.

Surface clay is defined by the Committee on Clay Sewer Pipe of the A.S.T.M. to be an unconsolidated, unstratified clay, occurring on the surface. Fire clay is a sedimentary clay, of low flux content, and is usually associated with coal measures. Shale is a thinly stratified, consolidated sedimentary clay with well-marked cleavage parallel to the bedding.

Clay is defined as an earthy or stony mineral aggregate consisting essentially of hydrous silicates of alumina, plastic when sufficiently pulverized and wetted, rigid when dry, and vitreous when burned at a sufficiently high temperature.

Manufacture.—The raw clay is delivered either from the clay pit directly, from storage bins, or from outdoor weathering piles, and is brought into the grinding room where it is discharged into the pans of the dry grinders or dry mills. The dry grinder or dry mill consists of a horizontal pan with a perforated cast-iron bottom. This pan is attached to a vertical drive shaft, and riding on the bottom are two heavy wheels with steel tires. These wheels, or mullers, are mounted as idlers on horizontal bearings and their revolution is produced by the turning of the pan.

After the clay is ground, it is pushed through slots in the bottom of the pan and is then discharged over a long screen having 10 to 16 meshes to the inch, set at an angle of about 45 deg. That portion of the ground clay which is fine enough passes through the screen into the ground-clay storage bin, while those particles which are still too coarse to pass

[1] A noteworthy example is the very detailed series of experiments upon flow of water in drain tile made for the U. S. Bureau of Public Roads, Dept. of Agriculture, by D. L. Yarnell (*Bull.* 854). No such exhaustive experiments upon flow in sewer pipe have ever been made.

through the screen are rejected and returned to the dry pan for further grinding.

From storage bins the ground clay is conveyed to the wet pans, which are similar to the dry grinders with the exception that the bottom is not perforated. The mullers are narrower and revolve at a somewhat higher rate of speed. Here the water and the clay are thoroughly mixed into a dough of uniform consistency.

The tempered clay is discharged upon a conveyor, which delivers it into the press, which operates under a pressure of about 120 lb. per square inch. The clay fills the mud cylinder of the press and is forced out at the lower end through a die which forms the sewer pipe.

The pipe is made or pressed with the socket or hub end downward and must be turned end for end to dry. The pipes of the larger sizes are turned in a cradle operated by machinery, because of their weight, while those of the smaller sizes are turned by the operators. The dry-room floor is constructed to insure a uniform temperature in order that the pipe may dry out and shrink uniformly without cracking.

From the dry room the pipe are taken into one of the beehive kilns. The floor of the kiln consists of large-sized firebrick so laid as to span a series of small smoke ducts. These small ducts are connected at the ends into a cross duct, which, in turn, discharges into the main flue leading to the stack or chimney. The gases from the fire rise to the top of the kiln, and thence pass downward through the sewer pipe, and out through the openings in the floor to the stack. In the setting of pipe in the kiln, small sizes are nested inside of larger sizes, but in such a way that there is complete circulation and draft around every piece. The pipes when set in the kiln are quite hard and dry.

The pipe, as it comes from the dry room, still contains about 3 per cent of moisture, and the burning, after the kiln is set and sealed, begins with very small fires so that this moisture can be driven off without cracking the pipe. If the heat is developed too rapidly, steam may be formed within the body of the pipe, which ruins it. A temperature not exceeding 300°F. is maintained for a number of hours. After this, the temperature is rapidly increased to about 1200°F., at which time the so-called oxidation period begins. There is in the clay considerable organic matter, especially in water-transported clays, together with certain mineral constituents which cannot stand the high degree of temperature required for vitrification. The firing during this stage must be carefully conducted so as to permit the release of gases through the body of the pipe. The oxidation temperatures range from about 1200 to 1400°F.

Following this period, the heat is rapidly raised to the sintering or vitrification temperature. The particles which are subject to melting by heat, flow and surround the more resistant particles, forming a dense, hard, coherent mass, a process which is known as vitrification. The

temperature to produce the vitrification varies somewhat with the character of the clay from which the pipe is made, but, in general, the finishing temperature is from 2000 to 2200°F.

When the temperature of the kiln has reached 2000°F. or more and the silica in the outer and inner surfaces of the pipe has been brought to the melting point, ordinary coarse salt (sodium chloride) is thrown upon the fires and the fire holes closed. The intense heat causes the sodium in the salt to combine chemically with the melted silica to form glass—the glaze.

The entire process of burning sewer pipe takes from 2 to 12 days, depending upon the size of the pipe.

The cooling of the kiln after the glazing has been completed is quite as important a process as the burning, because if the pipe cools too quickly the glaze may become cracked or crazed, or the pipe body cracked or air checked. The temperature of the kiln is gradually reduced by slowly opening the top and the kiln door.

Branches such as Y's and T's, are made from two pipes which are delivered to the branch maker from the pipe press. In the main pipe barrel there is cut, by means of a templet, a hole of a size equal to the outside diameter of the branch pipe. The branch pipe is cut in a form and then set into the hole in the barrel, after which clay is pressed by hand around the joint on the outside, thus joining the branch and the barrel. The inside of the branch is trimmed to the shape and contour of the barrel.

Other specials, as traps, curves and reducers, are usually cast in plaster of paris molds and permitted to remain therein for a day or two.

Shapes and Dimensions.—Originally, each manufacturer of vitrified pipe had his own standards, differing from every other in most particulars except internal diameters. In course of time, however, practice became more nearly standardized and two nearly uniform sets of dimensions came into general use for "standard" and "double-strength" sewer pipe, the latter being made only in 15-in. and larger sizes. Somewhat later the demand for larger and deeper sockets became sufficient to result in the so-called "deep and wide socket," which could be obtained upon either "standard" or "double-strength" pipe. There were then four generally recognized classes of sewer pipe, namely, standard, standard with deep and wide socket, double strength, and double strength with deep and wide socket.

Although dimensions and weights of pipe from various makers were not uniform, the figures given in Table 112 may be taken as a fair representation of the manufacturers' standards.

In an attempt to reduce the number of classes and patterns of pipe in use, the A.S.T.M. adopted (1924) a single class of sewer pipe, in which the thickness of shell is that of the manufacturers' "double-strength"

TABLE 112.—APPROXIMATE DIMENSIONS AND WEIGHTS OF SEWER PIPE

Eastern Clay Products Association, 1925

Diameter, inches	Single-strength pipe					Single-strength pipe, deep and wide socket					Double-strength pipe conforming, in general, to A S.T.M. standards					Double-strength pipe, deep and wide socket				
	Thickness of shell, inches	Depth of socket, inches	Annular space, inches	Weight per foot, pounds		Thickness of shell, inches	Depth of socket, inches	Annular space, inches [2]	Weight per foot, pounds		Thickness of shell, inches	Depth of socket, inches	Annular space, inches[1]	Weight per foot, pounds		Thickness of shell, inches	Depth of socket, inches	Annular space, inches	Weight per foot, pounds	
				E. C P. A.[2]	A. S. P. C.[3]				E. C. P. A.[1]	A S P. C.[2]				E. C. P. A.[2]	A. S. P. C.[3]				E. C. P. A.[2]	A. S. P. C.[3]
4	9/16	1 3/4	3/8	7 1/2	9	1/2	2	1/2	10	10	9/16	1 3/4 [1]	3/8	7 1/2						
5	5/8	2	13/32	11	12	5/8	2 1/2	5/8	12	12	5/8 [1]	2 [1]	13/32	11						
6	5/8	2 1/4	13/32	13	15	5/8	2 1/2	5/8	16	16	5/8	2 1/4 [1]	13/32	13						
8	3/4	2 1/2	1/2	20	23	3/4	2 3/4	5/8	25	25	3/4	2 1/2 [1]	1/2	20						
10	7/8	2 1/2	1/2	30	35	7/8	2 3/4	5/8	37	37	7/8	2 1/2	1/2	30						
12	1	2 1/2	1/2	42 1/2	45	1	3	5/8	45	45	1	2 1/2	1/2	42 1/2						
15	1 1/8	2 1/2	1/2	60	60	1 1/2	3	5/8	65	70	1 1/4	2 1/2	1/2	70	75	1 1/4	3	5/8	75	75
18	1 1/4	3	17/32	80	85	1 1/4	3 1/4	5/8	86	90	1 1/2	3	17/32	100	118	1 1/2	3 1/4	5/8	107.	118
20	1 3/8	3	17/32	100	100	1 3/8	3 1/2	5/8	105	115	1 5/8 [1]	3 [1]	17/32	125	138	1 3/8	3 1/2	5/8	135	138
21	1 1/2	3	9/16	112	120	1 1/2	3 5/8	5/8	130	130	1 3/4	3	19/32	135	148	1 3/4	3 5/8	5/8	148	148
22	1 5/8	3	19/32	122	130	1 5/8	3 3/4	5/8	145	145	1 7/8 [1]	3 [1]	19/32	155	157	1 5/8	3 3/4	5/8	165	157
24	1 5/8	3	19/32	140	140	1 5/8	4	5/8	150	150	2	3	19/32	175	190	2	4	5/8	190	190
27	2	3 1/2	1 1/16	195	224	...	...	..	...	...	2 1/4	3 1/2	1 1/16	215	265	2 1/4	4	3/4	260	
30	2 1/8	3 1/2	1 3/16	230	252	...	...	..	...	...	2 1/2	3 1/2	1 3/16	275	290	2 1/2	4	3/4	310	
33	2 1/4	4	1 3/16	300	310	...	...	..	...	...	2 5/8	4	1 3/16	340	335	2 5/8	5	1 1/4	335	
36	2 1/2	4	1 1/16	350	350	...	...	..	...	...	2 3/4	4	1 1/16	390	375	2 3/4	5	1 1/4	390	

[1] Not A.S.T.M. standards.
[2] Eastern Clay Products Association, 1925.
[3] American Sewer Pipe Company, 1916.

pipe for sizes in which double-strength pipe is made, and in which the dimensions of the socket are intermediate between those of the manufacturers' "standard" and the "deep and wide socket."

The dimensions of vitrified clay sewer pipe required by the A.S.T.M. specifications are given in Table 113. The form of the pipe and bell are shown in Fig. 104.

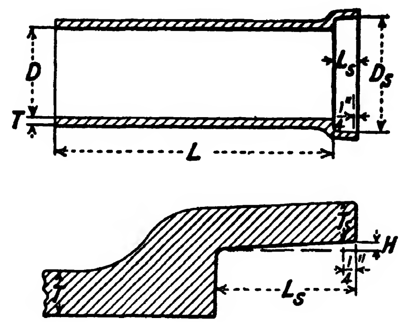


FIG. 104.—Form and dimensions of clay and concrete sewer pipes for A. S. T. M. standards as given in Table 113.

TABLE 113.—DIMENSIONS OF CLAY SEWER PIPE
American Society for Testing Materials

Internal diameter D, inches	Laying length L, feet	Inside diameter at mouth of socket D_s, inches[1]	Depth of socket L_s, inches	Minimum taper of socket H	Thickness of barrel T, inches	Thickness of socket T_s	Approximate weight per foot,[2] pounds
4	2	6	1½	1:20	9/16	The thickness of the socket ¼ in. from its outer end shall not be less than three-fourths of the thickness of the barrel of the pipe.	9
6	2	8¼	2	1:20	⅝		15
8	2, 2½, 3	10¾	2¼	1:20	¾		24
10	2, 2½, 3	13	2½	1:20	⅞		33
12	2, 2½, 3	15¼	2½	1:20	1		45
15	2, 2½, 3	18¾	2½	1:20	1¼		75
18	2, 2½, 3	22¼	3	1:20	1½		105
21	2, 2½, 3	26	3	1:20	1¾		145
24	2, 2½, 3	29½	3	1:20	2		185
27	2½, 3	33¼	3½	1:20	2¼		235
30	2½, 3	37	3½	1:20	2½		300
33	2½, 3	40¼	4	1:20	2⅝		350
36	2½, 3	44	4	1:20	2¾		385
39	2½, 3	47¼	4	1:20	2⅞		
42	2½, 3	51	4	1:20	3		

[1] When pipes are furnished having an increase in thickness over that given, the diameter of socket shall be increased by an amount equal to twice the increase of thickness of barrel.

[2] From "Tentative Standards" issued by Clay Products Association, 1925.

The A.S.T.M. standard of a single class of pipe has not generally been adopted and modification of the specifications is now (1928) under consideration.

There is considerable demand for thinner pipe than the "double-strength" (A.S.T.M.), and the Clay Products Association has issued a pamphlet of standards in which the A.S.T.M. dimensions are generally followed for "double-strength" pipe, and dimensions in Table 112 for "single-strength" ("standard") pipe, but with sockets of the same general shape as the A.S.T.M. pattern. The deep and wide sockets are still furnished by some manufacturers while others have stopped making them.

The joint room provided in sockets of the A.S.T.M. design is not sufficient to satisfy those engineers who demand a "deep and wide socket" and it seems unlikely that standardization will be accomplished upon the basis of a single class of pipe. At present some of the makers continue to produce their regular classes of pipe, as above stated, and it may be necessary to modify the A.S.T.M. requirements so as to cover the four classes, although it is to be hoped that standards may be adopted so that the dimensions of any class will be identical throughout the country.

Although the A.S.T.M. specifications extend to pipes 42 in. in diameter, the manufacturers' lists include no sizes larger than 36 in. Under present conditions, and in view of the concrete backing which would usually be needed for the larger sizes, vitrified pipe sewers larger than 27 or 30 in. in diameter are likely to cost more than monolithic concrete.

Physical Properties.—In accordance with the specifications of the A.S.T.M., clay sewer pipe must have the following qualities:

It must withstand internal hydrostatic pressure of 5 lb. per square inch for 5 min., 10 lb. for 10 min., and 15 lb. for 15 min., without showing leakage.

Sound pieces of pipe, with all edges broken, and thoroughly dried, must not absorb more than 8 per cent of their weight of water, after boiling for 5 hours.

The average crushing strength per foot of length, with the types of loading shown in detail in the specifications, must not be less than the amounts shown in Table 114.

Plain-end Pipe.—If desired, vitrified sewer pipe can be obtained with plain ends. This type of pipe is commonly called "ring pipe." The joints may be made by means of rings or collars slightly larger than the pipe, which are broken into pieces and imbedded in mortar around the ends of adjoining lengths of pipe. Such joints are neither as strong nor as tight as well-made socket joints, and unsupported plain-end pipe is, therefore, rarely used for sewers at the present time.

Plain-end pipe used with or without joint rings may also be employed as a liner and interior form, entirely surrounded by concrete, where

TABLE 114.—PHYSICAL TEST REQUIREMENTS OF CLAY SEWER PIPE

Internal diameter, inches	Average crushing strength, pounds per linear foot	
	Knife-edge and three-edge bearings	Sand bearings
4	1,000	1,430
6	1,000	1,430
8	1,000	1,430
10	1,100	1,570
12	1,200	1,710
15	1,370	1,960
18	1,540	2,200
21	1,810	2,590
24	2,150	3,070
27	2,360	3,370
30	2,580	3,690
33	2,750	3,930
36	3,080	4,400
39	3,300	4,710
42	3,520	5,030

greater strength is required than that of the pipe itself. This is substantially in accordance with the practice at Washington, D.C., as illustrated in Fig. 112, p. 388, although, in general, the pipe is not entirely surrounded by concrete except at the joint. A vitrified ring is not used. Such a sewer should be quite as good as one in which socket pipe were used.

CEMENT-CONCRETE PIPE

Material.—The materials used in cement-concrete pipe are portland cement, suitable aggregates, and water.

Manufacture.—Cement-concrete pipes are made in metal molds with a concrete mixture of suitable proportions and consistency, and well compacted by ramming (tamping process) or pressure (packerhead process). In either case the concrete is subjected to great pressure, after which the molds are removed and the pipes are cured from 48 to 72 hours in a room kept warm and damp with low-pressure steam, a water spray, or both. High temperature accelerates the hardening of the concrete. Curing in such a manner as to avoid the evaporation of the water needed for hydration of the cement is of great importance. It is stated that web-like markings on the surface of the pipe are indicative of proper hydration and curing.

Plain concrete pipe may also be made by the centrifugal process, in which a measured quantity of wet concrete is placed in a mold which is then rotated rapidly about its axis. The centrifugal force packs the concrete solidly against the form, so that the pipe can be removed at once for curing. This process gives a dense, smooth pipe, but is not well adapted to forming a socket joint, and collars have been used for jointing; it can also be employed to advantage for pipes of greater length than those called for by the A.S.T.M. specifications. The centrifugal process has not yet been used much in the United States, except for pipes of reinforced concrete.

Dimensions.—The A.S.T.M. standard dimensions for cement-concrete sewer pipe are the same as those for vitrified-clay pipe (Table 113, p. 368), except that the laying lengths for 4- and 6-in. pipes may be either 2 or 2½ ft., instead of 2 ft. only, as for the vitrified pipes, and that the thickness of barrel for some of the sizes above 21 in. is slightly greater for concrete than for clay pipe.

At the present time (1928) these pipes are being made commercially in 4- to 24-in. sizes. Few, if any, makers are equipped to furnish larger than 24-in. plain concrete pipe.

Physical Properties.—In accordance with the specifications of the A.S.T.M., cement-concrete sewer pipe must have the following qualities:

It must withstand internal hydrostatic pressure of 5 lb. per square inch for 5 min.; 10 lb. for 10 min.; and 15 lb. for 15 min., without showing leakage.

Sound pieces of pipe, with all edges broken, and thoroughly dried, must not absorb more than 8 per cent of their weight of water, after boiling for 5 hours.

The average crushing strength per foot of length, with the types of loading shown in detail in the specifications, must not be less than the amounts shown in Table 114.

Chemical Tests and Requirements.—The A.S.T.M. specifications prescribe that:

4. The consumer or purchaser may prescribe in advance special chemical requirements in cases where sewage, industrial wastes or ground waters have marked acid or alkaline character, or are of abnormally high temperatures. He may make use of chemical analysis of the pipe materials to ascertain whether these special requirements are met. The presence of deleterious materials causing slaking or disintegration shall be cause for rejection.

This requirement also appears in the specifications for vitrified-clay pipes, but such pipes are rarely affected by acids or alkalies, and this requirement is therefore of major importance only for cement pipes in certain localities or under certain conditions as to character of sewage.

In Technologic Paper 214[1] of the U. S. Bureau of Standards, the following conclusions appear:

1. The use of concrete tile in soils containing alkali salts of the sulphate type in considerable quantities is hazardous, in view of the fact that the best quality of drain tile has been disintegrated during an exposure of less than 6 years.

2. Porous or permeable tile, due to the use of lean mixtures, or relatively dry consistences, are subject to disintegration in sulphate waters of relatively low concentrations.

3. Dense tile of the best quality, exposed to sulphate waters, are under certain conditions subject to disintegration, depending upon concentration of salts in seepage water, and alkali and moisture conditions in the soil immediately surrounding the tile.

4. Disintegration may be manifested in sulphate waters by physical disruption caused by expansion resulting from the crystallization of salts in the pores, but it is primarily due to chemical action between the salts in solution and the constituents of the cement. In the case of dense tile of low permeability exposed to sulphate waters, disintegration may occur at or just inside the surface skin and progress into the wall of the tile.

5. A thin outer skin of apparently unaffected concrete is apparent in the case of disintegration of the best quality of tile. While this relatively thin layer may be either immune to attack by salts in solution, or perhaps be very slowly attacked, results indicate that this carbonized coating is not in itself waterproof, and alkali water may pass through this coating into the mass.

6. In sulphate waters with a concrete or mortar of given quality the disintegrating effect seems to vary with the concentration of the solution.

7. Tile made by the process commonly used, which allows the removal of forms immediately after molding, are more susceptible to disintegration where exposed to sulphate soils or waters than are tile made of wetter consistency which requires their retention in the molds for a period of hours.

8. The use of hand-tamped tile of plastic consistency, such that the jacket can be removed immediately after molding, can not be recommended for use in sulphate soils and waters. Tile of plastic consistency, molded in the packer-head type of machine, are more resistant to alkali action than the hand-tamped tile, but the best quality of packer-head machine tile have been affected in waters of high salt concentration.

9. Steam-cured tile show no greater resistance to alkali action than tile which are cured by systematic sprinkling with water.

10. Tile made of sand cement have less resistance to alkali action than tile made of portland cement of the same proportions.

11. The tar coating as used was not effective in preventing the absorption of alkali water into the walls of the tile.

12. The cement-grout coating is not effective in preventing the absorption of alkali water.

[1] "Durability of Cement Drain Tile and Concrete in Alkali Soils," Third Progress Report, 1919–1920.

13. No advantage was found in introducing ferrous sulphate into the cement mixture. The use of this material resulted in a reduction of the crushing strength of the tile.

14. If cement drain tile are to be used in soils and waters containing 0.1 per cent or more of salts of the sulphate type, their installation should be preceded by an examination of sub-surface alkali conditions. Decision as to the advisability of using cement drain tile should be based upon thorough examination of sub-surface conditions and quality of product which it is proposed to use, and upon a comparison of the data so obtained with the data presented in this report. Such an examination may indicate portions of an area where the use of cement tile should be avoided. Quantities of alkali salt present can be determined by chemical analyses of the soils. Concentrations to which tile may be exposed can be determined by chemical analyses of the ground waters. In drawing conclusions, allowance must be made for the constantly changing concentrations of alkali in the seepage water, which, at times, may differ as much as several hundred per cent in adjacent areas. There can be no assurance that the concentrations found at any time are the maximum to which the tile will ever be exposed. Quality of cement drain tile can best be measured by permeability tests. With other conditions equal, tile of lowest permeability will be the most durable. There appears to be little definite relation between permeability and the related factors of porosity, absorption, and density.

While these tests were made on drain tile for which the requirements are less strict than for sewer pipe, they indicate the possibility of similar damage to sewer pipe under the same conditions.

Similar disintegration of cement-concrete pipe has been experienced where the sewage contains or may generate an excessive amount of sulphuric acid; and it has been claimed that, in certain cases, acids derived from humus in the soil have produced like effects, although other tests have shown concrete pipes in acid soils to be in good condition after a considerable term of years.

Further discussion of the disintegration of concrete will be found on page 451, Chap. XII.

REINFORCED-CONCRETE PIPE

For sizes above 24 in. diameter and up to 108 in., reinforced-concrete pipe may be used. Such pipe may be field-cast, machine-made, or made by the centrifugal process. Field-cast pipe are manufactured on or near the site of the work, and require the use of inner and outer steel forms. Machine-made pipe must be made in a permanent plant where the concrete can be subjected to great pressure, as in the case of plain concrete pipe. This process has been used only for sizes up to 54- or 60-in. In the centrifugal process only an outer form is required and the concrete is compacted by centrifugal force.

The larger sizes of pipes must be compared with monolithic concrete or brick sewers and do not come within the definition of pipe sewers adopted in this chapter, which limits the size to 42-in. diameter.

Socket and spigot joints of the ordinary type may be used for reinforced-concrete pipe. Most of the makers have adopted a joint of the mortise and tenon pattern, in which there is no enlargement of the external diameter, or diminution of the internal diameter, at the joint.

Reinforced-concrete pipe may be made of sufficient strength and tightness to withstand a considerable internal pressure and, therefore, has been used in some cases for force mains and inverted siphons.

Minimum thicknesses, amounts of steel, and ultimate test loads for reinforced-concrete pipe up to 72 in. in diameter have been tentatively adopted by the American Concrete Institute,[1] and are given in Table 115.

TABLE 115.—MINIMUM DIMENSIONS AND ULTIMATE STRENGTH REQUIREMENTS OF REINFORCED-CONCRETE SEWER PIPE

Tentative Standard Specifications, American Concrete Institute, 1925

Internal diameter of pipe	Minimum dimensions for				Ultimate load in pounds per linear foot of pipe (28 days)	
	Field-cast pipe		Shop-made pipe			
	Thickness, inches	Minimum area steel, square inch per linear foot	Thickness, inches	Minimum area steel, square inch per linear foot	Three-edge bearing	Sand bearing
24	3	0.065	2½	0.15	3,000	4,500
27	3	0.065[1]	2⅝	0 18	3,300	4,950
30	3½	0.085	2¾	0.21	3,600	5,400
33	3¾	0.105	2¾	0.24	3,900	5,850
36	4	0.125	3	0.36	4,200	6,300
42	4½	0.150	3⅜	0.42	4,700	7,050
48	5	0.210	3¾	0.46	5,100	7,650
54	5½	0.250	4⅛	0.50	5,500	8,250
60	6	0.290	4½	0.54	5,800	8,700
66	6½	0.320	4¾	0.58	6,000	9,000
72	7	0.360	5	0.62	6,200	9,300

NOTE: Sizes below heavy line have reinforcing near both inner and outer faces.

[1] Correctly copied, but probably should be 0.075.

As an indication of the strength of reinforced-concrete pipe to sustain external loads without the assistance of concrete cradles or other additional construction, Table 116 has been prepared. In computing the figures in this table, it has been assumed that the pipe can carry working loads equal to one-half the ultimate loads given in Table 115, before visible cracks will appear, since the American Concrete Institute specifi-

[1] *Proc. A. C. I.*, 1925; **21**, 584.

TABLE 116.—COMPUTED DEPTH OF FILL THAT CAN BE SUPPORTED BY REINFORCED-CONCRETE PIPE WHICH COMPLIES WITH AMERICAN CONCRETE INSTITUTE SPECIFICATIONS

Size of pipe, inches	Load pipe can support without visible cracks.[1] American Concrete Institute Specifications	Width of trench equals outside diameter plus 2 ft. 0 in.	Depth of fill above crown for load on pipe equal to load at first crack		Width of trench equals outside diameter plus 1 ft. 0 in	Depth of fill above crown for load on pipe equal to load at first crack		Width of trench equals outside diameter of pipe	Depth of fill above crown for load on pipe equal to load at first crack	
			Saturated top soil w = 110 lb./cu. ft	Saturated clay, w = 130 lb./cu. ft.		Saturated top soil, w = 110 lb./cu. ft.	Saturated clay, w = 130 lb./cu. ft.		Saturated top soil, w = 110 lb./cu. ft.	Saturated clay, w = 130 lb./cu. ft.
24	2,250	4′- 6″	5′- 5″	4′-3″	3′- 6″	8′- 1″	5′-11″	2′- 6″	28′-6″	10′- 9″
27	2,475	4′- 9″	5′- 8″	4′-5″	3′- 9″	8′- 2″	6′- 1″	2′- 9″	19′-11″	10′- 2″
30	2,700	5′- 1″	5′- 9″	4′-6″	4′- 1′	7′-11″	6′- 0″	3′- 1″	15′- 2″	9′- 3″
33	2,975	5′- 4½″	5′-10″	4′-7″	4′- 4½″	7′-10″	6′- 0″	3′- 4½″	13′- 4″	8′-10″
36	3,150	5′- 8″	5′-11″	4′-8″	4′- 8″	7′- 9″	6′- 0″	3′- 8″	12′- 4″	8′- 6″
42	3,525	6′- 3″	6′- 2″	4′-9″	5′- 3″	7′- 6″	5′-10″	4′- 3″	10′- 7″	7′- 9″
48	3,825	6′-10″	5′-10″	4′-7″	5′-10″	7′- 2″	5′- 7″	4′-10″	9′- 6″	7′- 2″
54	4,125	7′- 5″	5′- 8″	4′-6″	6′- 5″	6′-10″	5′- 5″	5′- 5″	8′- 9″	6′- 9″
60	4,350	8′- 0″	5′- 6″	4′-5″	7′- 0″	6′- 6″	5′- 2″	6′- 0″	8′- 1″	6′- 3″
66	4,500	8′- 7″	5′- 3″	4′-2″	7′- 7″	6′- 2″	4′-11″	6′- 7″	7′- 4″	5′- 9″
72	4,650	9′- 2″	5′- 0″	4′-1″	8′- 2″	5′- 9″	4′- 8″	7′- 2″	6′-10″	5′- 5″

[1] Taken as one-half the specified "ultimate load" with sand bearing.

cations require that "the test specimens shall show no clearly visible cracks caused by the application of the load extending the full length of the pipe, when tested to one-half the ultimate load." It was also assumed that the loads from the backfill in the trench were distributed over the top quadrant of the pipe, and that their amounts could be computed by the method of Marston and Anderson, explained in Chap. XIII. The advantage of narrower trenches in carrying appreciable amounts of the weight of backfill, particularly in the case of the smaller pipes, is obvious.

These figures show that, except for the smaller pipe in very narrow trenches, reinforced-concrete pipes fulfilling the American Concrete Institute specifications have not sufficient strength to carry the weight of the backfill in deep trenches. Concrete cradles or other constructions to provide additional strength are likely to be required, therefore, in many cases.

CAST-IRON, STEEL, AND WOOD PIPES

Cast-iron Pipe.—Where a sewer is under considerable internal pressure, as in force mains and inverted siphons, cast-iron pipe has generally been used, although in a few instances steel or wood-stave pipes have been employed, and recently the use of reinforced-concrete pipe has become common.

Cast-iron pipe sewers have also been employed in some cases where the quantity of water in the ground is large and where it is particularly desirable to limit the leakage of ground water into sewers as much as possible. The relative ease of making water-tight joints with cast-iron pipe, as compared to sewer pipe, as well as the imperviousness of the iron pipes themselves, has been largely responsible for this use.

Cast iron is also commonly used for the outfall or discharge end of a pipe sewer where it empties into a body of water, owing to its greater resistance to the action of flowing water, ice, drift, and the like. The submerged outfalls in Boston harbor are illustrations of such use, and similar outfalls in Lake Erie at Cleveland are examples of the use of reinforced-concrete pipe.

Cast-iron pipe in water works systems commonly grows rougher with age, owing to the formation of tubercles on the interior surface. This is particularly the case when the water is soft. The effect of such tuberculation is to reduce the capacity of the pipes. Whether or not similar conditions develop in cast-iron pipe when used for sewage does not appear to have been established. It is probable that a sufficient coating of grease may accumulate upon the pipes to prevent tuberculation; and that reduction in capacity, as compared with new pipe, is more likely to result from accumulations of grease and sewage solids than from tuberculation of the pipe. A 42-inch cast iron sewage force main at

Baltimore, Md., after 15 years' use, had accumulated a coating of grease and slime about one inch thick, but when this was removed the pipe was found to be free of tubercles. The material of the coating, when dried, was found to contain 34 per cent of iron.[1]

Standard sizes of cast-iron pipe range from 4 to 20 in. in diameter, by increments of 2 in., and from 24 to 60 in. in diameter by increments of 6 in. The 18-in. size, however, is rarely used in water-works practice and some foundries might refuse to supply it. The 14-in. size is becoming somewhat uncommon. The 72- and 84-in. sizes, while included in the tabulation, can be obtained only on special order and few foundries are equipped to furnish them.

Standard thicknesses and weights for cast-iron water pipe, in accordance with the specifications of the American Water Works Association, are given in Table 117. The lightest standard weight is usually amply strong for sewerage use, unless unusual loads or stresses due to settlement are to be expected. Sometimes the prices charged for Class *A* pipe are enough higher than those for heavier pipes so that there is no economy in using the lighter weights.

Centrifugally cast-iron pipe is also to be had in sizes up to 20 in. This pipe is cast in a revolving mold and without a core. It is thinner than the standard sand-cast pipe. It is claimed that casting by the centrifugal process results in a denser and more homogeneous metal in the wall of the pipe, so there is no sacrifice in strength, although the pipe is thinner; and also that corrosion is likely to be less. The cost per foot is generally 5 to 10 per cent less than for sand-cast pipe.

Steel Pipe.—For conditions similar to those in which cast-iron pipe might be used, steel pipe would also be applicable. The longitudinal joints in this pipe may either be welded, riveted or made with a lock bar; the circumferential joints are usually riveted, though other types are now available.

In some cases, especially where soft ground is to be crossed, making it desirable to use a light-weight pipe, steel pipe may be more advantageous than cast iron. On the other hand, it is more susceptible to corrosion, so that it is especially important that the metal be coated as thoroughly as possible; and the riveted joints are objectionable on account of the resulting resistance to flow, and also as favoring the lodgment of sewage solids.

Wood Pipe.—Outfall sewers have been made of wood-stave pipe in a few cases. This material has the advantage, as compared with cast-iron or steel, that it does not corrode nor rot when always submerged; the interior surface is smooth, and the carrying capacity does not decrease with age. It has the disadvantage that it must be weighted and anchored to the bottom and that the bands may rust away rapidly, after which the pipe will fall to pieces.

[1] *Eng. News-Rec.* 1928; **100,** 360.

TABLE 117.—STANDARD THICKNESSES AND WEIGHTS OF CAST-IRON PIPE
American Waterworks Association

Nominal Diam., in.	Class A, 100-ft. head, 43 lb. pressure			Class B, 200-ft. head, 86 lb. pressure			Class C, 300-ft. head, 130 lb. pressure			Class D, 400-ft. head, 173 lb. pressure		
	Thickness, in.	Weight per Ft.	Weight per Length	Thickness, in.	Weight per Ft.	Weight per Length	Thickness, in.	Weight per Ft.	Weight per Length	Thickness, in.	Weight per Ft.	Weight per Length
4	0.42	20.0	240	0.45	21.7	260	0.48	23.3	280	0.52	25.0	300
6	0.44	30.8	370	0.48	33.3	400	0.51	35.8	430	0.55	38.3	460
8	0.46	42.9	515	0.51	47.5	570	0.56	52.1	625	0.60	55.8	670
10	0.50	57.1	685	0.57	63.8	765	0.62	70.8	850	0.68	76.7	920
12	0.54	72.5	870	0.62	82.1	985	0.68	91.7	1,100	0.75	100.0	1,200
14	0.57	89.6	1,075	0.66	102.5	1,230	0.74	116.7	1,400	0.82	129.2	1,550
16	0.60	108.3	1,300	0.70	125.0	1,500	0.80	143.8	1,725	0.89	158.3	1,900
18	0.64	129.2	1,550	0.75	150.0	1,800	0.87	175.0	2,100	0.96	191.7	2,300
20	0.67	150.0	1,800	0.80	175.0	2,100	0.92	208.3	2,500	1.03	229.2	2,750
24	0.76	204.2	2,450	0.89	233.3	2,800	1.04	279.2	3,350	1.16	306.7	3,680
30	0.88	291.7	3,500	1.03	333.3	4,000	1.20	400.0	4,800	1.37	450.0	5,400
36	0.99	391.7	4,700	1.15	454.2	5,450	1.36	545.8	6,550	1.58	625.0	7,500
42	1.10	512.5	6,150	1.28	591.7	7,100	1.54	716.7	8,600	1.78	825.0	9,900
48	1.26	666.7	8,000	1.42	750.0	9,000	1.71	908.3	10,900	1.96	1,050.0	12,600
54	1.35	800.0	9,600	1.55	933.3	11,200	1.90	1,141.7	13,700	2.23	1,341.7	16,100
60	1.39	916.7	11,000	1.67	1,104.2	13,250	2.00	1,341.7	16,100	2.38	1,583.3	19,000
72	1.62	1,283.4	15,400	1.95	1,545.8	18,550	2.39	1,904.2	22,850			
84	1.72	1,633.4	19,600	2.22	2,104.2	25,250						

The above weights are per length to lay 12 ft., and include weight of bell.

STRENGTH OF SEWER PIPES TO CARRY EXTERNAL LOADS

The theoretical analysis of the strength of pipe under external loads is that of thin elastic rings, and takes various forms under different assumptions regarding the loading. An explanation of it is given by Prof. A. N. Talbot, in *Bull.* 22, *Eng. Exp. Sta. Univ. Illinois*, in which he reports tests of cast-iron and reinforced concrete culvert pipe. Marston and Anderson have given the results of an analysis in which the weight of the pipe, as well as that of the backfilling, is taken into consideration. For all practical purposes, three assumed conditions of loading will be sufficient to guide the engineer, *viz.*, concentrated loads at the top and bottom of the vertical diameter of the pipe, uniformly distributed vertical loads above and below the horizontal diameter, and uniformly distributed loads on the top quarter and bottom quarter of the circumference of the pipe.

The bending moment at the top and bottom of a pipe is the product of the total load per unit of length of pipe multiplied by the diameter of the center of the shell of the pipe, multiplied by 0.159 for a concentrated load; by 0.0625 for a load uniformly distributed over the entire width of the pipe; and by 0.0845 for a load uniformly distributed over the top quarter of the circumference, the bottom reaction in each case being distributed similarly to the load.

The factors for obtaining the bending moments at the ends of the horizontal diameters under these conditions are 0.091, 0.0625, and 0.077, respectively.

Laboratory Tests.—The requirements for crushing strength of sewer pipe under test, stipulated by the A.S.T.M., are given in Table 114, p. 370.

The requirements of the A.S.T.M. as to methods of applying the loads are as follows:

Knife or Two-edge Bearings.—The pipe to be tested shall be supported by a metallic knife bearing 1 in. wide and extended from a point just back of the socket to the spigot end of the pipe. Before the pipe is placed, a fillet of plaster of paris and sand 1 in. wide, and thick enough to compensate for all the inequalities of the pipe barrel, shall be cast on the surface of the knife-edge bearing. The pipe shall be placed upon the fillet while the plaster of paris is still somewhat plastic. The load shall be applied through an upper knife bearing of the same size and length as the lower bearing. A plaster-of-paris fillet 1 in. wide shall be cast along the length of the crown of the pipe to equalize the lower bearing[1] before the upper one is brought into contact.

Both of the bearings shall be sufficiently rigid to transmit and receive uniform loads throughout their lengths without deflection, and shall be so attached to the machine as to transmit and receive the maximum stresses produced by the tests without lost motion, vibration, or sudden shock.

Three-edge Bearings.—When three-edge bearings are used, the ends of each specimen of pipe shall be accurately marked in halves of the circumference prior to the test.

The two lower bearings shall consist of two wooden strips with vertical sides, each strip having its interior top corner rounded to a radius of approximately ½ in. They shall be straight, and shall be securely fastened to a rigid block with their interior vertical sides 1 in. apart.

The upper bearing shall be a wooden block, straight and true from end to end.

The test load shall be applied through the upper bearing block in such a way as to leave the bearing free to move in a vertical plane passing midway between the lower bearings.

In testing a pipe which is "out of straight," the lines of the bearings chosen shall be from those which appear to give most favorable conditions for fair bearings.

Sand Bearings.—When sand bearings are used, the ends of each specimen of pipe shall be accurately marked prior to the test in quarters of the circumference. Specimens shall be carefully bedded, above and below, in sand, for one-fourth the circumference of the pipe measured on the middle line of the barrel. The depth of bedding above and below the pipe at the thinnest points shall be one-half the radius of the middle line of the barrel.

The sand used shall be clean, and shall be such as will pass a No. 4 screen.

The top bearing frame shall not be allowed to come in contact with the pipe nor with the top bearing plate. The upper surface of the sand in the top bearing shall be struck level with a straightedge, and shall be covered

[1] That is, the lower of the two bearing surfaces at the crown of the pipe.

with a rigid top bearing plate, with lower surface a true plane, made of heavy timbers or other rigid material, capable of distributing the test load uniformly without appreciable bending. The test load shall be applied at the exact center of this top bearing plate, in such a manner as to permit free motion of the plate in all directions. For this purpose a spherical bearing is preferred, but two rollers at right angles may be used. The test may be made without the use of a testing machine, by piling weights directly on a platform resting on the top bearing plate, provided, however, that the weights shall be piled symmetrically about a vertical line through the center of the pipe, and that the platform shall not be allowed to touch the top bearing frame.

The frames of the top and bottom bearings shall be made of timbers so heavy as to avoid appreciable bending by the side pressure of sand. The interior surfaces of the frames shall be dressed. No frame shall come in contact with the pipe during the test. A strip of cloth may, if desired, be attached to the inside of the upper frame on each side, along the lower edge, to prevent the escape of sand between the frame and the pipe.

Comparison of Test Results with Strength Developed by Pipe as Laid. A comparison of the requirements as to strength developed under different methods of testing indicates, as has been found to be a fact, that the two- and three-edge methods give substantially equal results, while the sand-bearing method gives results approximately 50 per cent higher.

Professor Anson Marston's experiments have shown that tests with sand bearings show with substantial accuracy the strength which will actually be developed by the pipe in the ground, when the "ordinary" method of laying is employed.

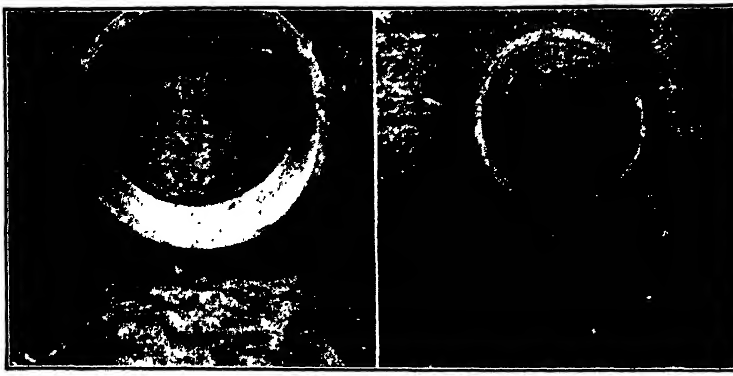
Strength of Pipe in the Ground.—The relation between the strength of pipe as developed by laboratory test and the amount which will be developed when the pipe is laid in the ground, has been found by Marston[1] to depend upon the character of the bearing between the pipe and the supporting earth. This means not only the earth upon which the pipe is laid (bottom of trench) but that on either side (walls of trench).

If the trench is excavated with a flat bottom, upon which the pipe is laid, the resulting condition will approximate that of the two-edge or three-edge laboratory test. The pipe cannot be expected to develop more than 80 per cent of its strength as shown by sand-bearing tests. Fig. 105 illustrates a pipe laid in this way. It should never be allowed and is, therefore, called the "impermissible" pipe laying method.

"Ordinary" pipe laying requires that the bottom of the trench be shaped to fit the pipe for a bearing arc of 60 to 90 deg. The backfill is placed loosely over and around the pipe. Figures 106 and 107 show pipe laid by this method. They may be expected to develop just about their test strength as shown by sand-bearing tests.

[1] Second Progress Report on Culvert Pipe Investigations, 1915–1921.

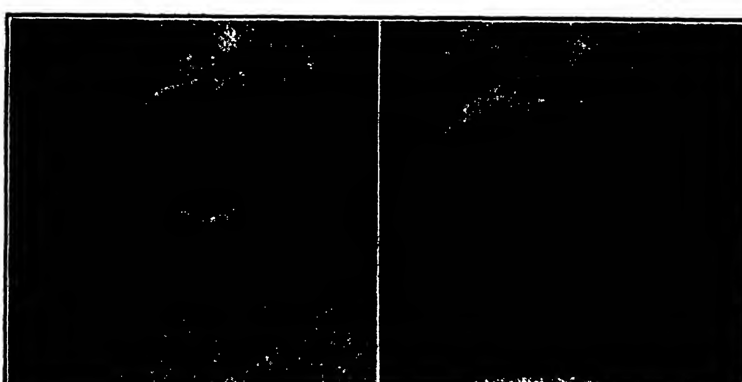
"First class" pipe laying requires the pipe to be especially well bedded for at least 90 deg. of its circumference, and the back-filling well tamped up to the level of the top of the pipe, thus ensuring a firm bearing between the sides of the pipe and the walls of the trench. Figure



FIGS. 105 and 106.—"Impermissible" and "ordinary" methods of laying sewer pipe.

108 shows a pipe laid in this way, developing about 120 per cent of the strength shown in laboratory tests with sand bearings.

By the use of a concrete cradle under the pipe and extending to the height of about one-third the diameter, a strength from 80 to 100 per



FIGS. 107 and 108.—"Ordinary" and "first-class" me thods of laying sewer pipe.

cent greater than the test strength of the pipe with sand bearings can be developed. Such a concrete cradle is shown in Fig. 109. This conclusion was based upon the tests of vitrified pipes up to 24 in. in diameter.

For the photographs reproduced in Figs. 105 to 109 the authors are indebted to Prof. Anson Marston.

Further experiments upon the value of concrete cradles for supporting conduits were reported in 1926.[1] These tests were made upon reinforced concrete pipes from 24 to 84 in. in diameter. The more important conclusions derived from them may be summarized as follows:

(1) The supporting strength of pipe before cracking may be increased 50 to 100 per cent by the use of properly designed cradles. This percentage of increase with a particular cradle will vary with the quality of the pipe and is higher for the weaker and lower for the stronger pipe.

FIG. 109.—Sewer pipe supported in concrete cradle.

(2) The use of a cradle which has a thickness under the pipe of one-fourth the nominal diameter of the pipe (¼ d) and which extends up the sides to a height of one-fourth the outside diameter of the pipe (¼ d') should increase the supporting strength about 75 per cent.

(3) Decreasing the proportional thickness of the cradle under the pipe and its height at the sides each reduce the effectiveness of the cradle.

(4) It is doubtful if it will be economically advisable to use reinforcing in a cradle with a thickness less than ¼ d under the pipe and a height at the sides of less than ½ d' for small pipe or ⅓ d' for large pipe. Reinforcing appears to be of value only in that portion of the cradle under the pipe. None of the cradles tested developed a visible fracture or crack except under the pipe. Properly designed and constructed reinforced

[1] SCHLICK, W. J., and J. W. JOHNSON, "Concrete Cradles for Large Pipe Conduits," *Bull.* 80, Iowa State College, 1926.

cradles of these dimensions should increase the cracking strength of the pipe 75 to 100 per cent.

Relation between Actual Supporting Strength of Pipe and Laboratory Test Strength.—Data upon which to base an estimate of the relation between the actual earth load carried by a pipe and the test strength, are contained in *Bull.* 76 of the Engineering Experiment Station, Iowa State College.[1] In this bulletin it is stated that the ratio of actual supporting strength to three-edge bearing laboratory test strength was approximately 2.6 at appearance of first crack. Without the help of the active side pressure the ratio would have been about 1.9.

STRENGTH OF SEWER PIPE TO WITHSTAND INTERNAL PRESSURE

Stress.—The bursting stress due to the internal pressure upon pipe with a thin wall is indicated by the formula

$$s = \frac{pr}{t}$$

in which

s = tension in shell of pipe, pounds per square inch
p = pressure of water in the pipe, pounds per square inch
r = radius of the pipe, inches
t = thickness of the pipe, inches

Sewers are seldom subjected to internal pressure of more than a few feet of water; consequently their strength in tension is not usually a matter of much moment.

Tests.—In 1890 tests were made at the Rose Polytechnic Institute, Terre Haute, Ind., by Prof. M. A. Howe, on pipe from 15 manufacturers. The results were published[2] and are summarized in Table 118. Three different methods were used in making the hydrostatic tests. The first and second caused a pressure to be exerted upon the ends of the pipe, while in the third no pressure was brought to bear on the ends, which were closed with leather cups. The tests showed consistently that the third method gave higher results, and it was considered more reliable by Howe. The other methods gave lower results because of stresses due to end pressure. The averages of tensile strengths for the different brands of pipe varied from 265.6 to 1081.8 lb. per square inch, while the general average of all results was 600.4 lb. The minimum recorded gage pressure was 12 lb. per square inch; the minimum tensile strength 68 lb. per square inch; the maximum tensile strength 1,825 lb., each of these being for a single test.

[1] Spangler, M. G., "A Preliminary Experiment on the Supporting Strength of Culvert Pipes in an Actual Embankment."

[2] *Jour. Assoc. Eng. Soc.*, 1891; 10, 283.

TABLE 118.—HYDROSTATIC TESTS OF VITRIFIED CLAY PIPE MADE AT ROSE POLYTECHNIC INSTITUTE BY PROF. M. A. HOWE

Designation of manufacturer		*A*	*B*	*C*	*D*	*E*	*F*	*G*	*H*	*I*	*J*	*K*	*L*	*M*	*N*	*P*
Average tensile strength by method	1	.	245.8	476.0	376.0											
	2	582 8	359.5	424.0	455.7	470.8	727.3	271 0								
	3	814.0	616.3	435.8	744.7	. .	. .	252.0	665.3	427.4	547.7	647.7	939.0	943.5	1,081.8	618.6
No. of tests		8	12	8	13	1	3	7	12	7	6	13	6	4	5	11
Average tensile strength (lb. per sq. in.)		669.5	407.2	442.9	582.9	470.8	727.3	265.6	665.3	427.4	547.7	647.7	939.0	943.5	1,081.8	618.6
Minimum gage pressure (lb. per sq. in)		50	28	47	25	62	125	12	76	42	63	59	144	139	85	37
Minimum tensile strength (lb. per sq. in.)		265	142	261	81	241	714	68	253	231	329	288	800	836	530	192
Maximum tensile strength (lb. per sq. in.).		1,054	775	580	921	988	745	507	1,023	587	751	1,099	1,093	1,095	1,825	1,086

In the hydrostatic tests the color of the fracture, with hardly an exception, was the criterion of strength, each class having its particular color corresponding to the greatest strength.

Tests were made at the Royal Testing Laboratory in Berlin, by Messrs. Burchartz and Stock, and their results for 1896–1904 inclusive were published.[1] The mean minimum, maximum and average values for resistance against internal pressure are given in Table 119. The average water pressure was 13.9 atmospheres or 204 lb. per square inch, and the average ultimate tensile strength 650 lb. per square inch. The minimum pressure was 5.3 atmospheres, equivalent to 78 lb. per square inch, or 183 ft. head of water. These results from German pipe are higher than those obtained by Howe from American pipe.

TABLE 119.—MEAN MINIMUM, MAXIMUM AND AVERAGE RESISTANCE TO INTERNAL PRESSURE OF GERMAN VITRIFIED-CLAY PIPE[1]

Average internal diameter, inches	Average thickness of shell, inches	Number of tests	Internal water pressure, pound per square inch			Ultimate average tangential stress, pounds per square inch
			Average minimum	Average maximum	Total average	
2	0 72	3	353	412	384	490
3	0.80	3	220	271	240	490
4	0 68	11	135	370	196	830
6[2]	0.80	23	122	360	253	850
8	0 92	13	215	356	263	1,120
10	0 96	2	88	118	103	600
12	1 04	19	99	248	175	920
16	1.12	2	138	143	141	910
18	1.28	8	104	176	137	920
20	1.44	4	132	179	156	1,040
24	1.72	7	78	126	106	720
28	1 88	4	104	150	128	910
32	1 96	14	79	132	113	1,120
Average... ..	. .	..	...	...	204	850

Strength of Joints.—Although vitrified pipe may be manufactured of sufficient strength to withstand hydrostatic pressures of 100 lb. per square inch or more, the strength of the joint is likely to be much less. There are few published data on this subject. Tests were made by Professor Howe at the Rose Polytechnic Institute on natural cement joints, and while his results are not indicative of the strength to be expected from portland cement joints, it is interesting to note that the

[1] BURCHARTZ and STOCK, *Eng. Record,* 1906; **54,** 190.

[2] Includes pipes 6.2 and 6.4 in. in diameter.

pressure withstood by the joint was considerably less than that which the pipe itself could withstand. Recent tests by Prof. S. E. Dibble at the Carnegie Institute of Technology indicate that portland cement mortar joints leak at internal pressures of 5 to 15 lb. per square inch. Tests made by the Philadelphia Division of Sewers on three jointing compounds show that joints made of these materials in 6- to 12-in. pipe will successfully withstand internal pressures of 25 to 30 lb. per square inch, reaching a maximum of 85 lb. for a 12-in. pipe (joint did not leak but pipe broke) and of 110 lb. for a 6-in. pipe. After testing, the specimens were buried for a year, some in sandy loam, others in wet marsh mud. They were then taken up and retested with similar results. Chemical tests of the jointing materials indicated no deterioration during this treatment.

PRACTICAL DEDUCTIONS FROM TESTS AND EXPERIENCE

The investigations described earlier in this chapter show that the loads coming upon pipe are frequently so great that the construction of a tight sewer under such conditions calls for good, intelligent workmanship. Experiments show that the elongation of the horizontal diameters of cement and clay pipe do not ordinarily exceed 0.04 in. under breaking loads. It is practically impossible to ram earth around the sides of a pipe so firmly that it will prevent such an insignificant movement and where the pipe is liable to be exposed to dangerous loads, it is necessary to use pipe of exceptionally high strength, bed it in a cradle of concrete, or use some other material for the sewer. It is evident from what has been said about the manufacture of clay and cement pipe that their tensile strength must be somewhat uncertain and that it is dangerous to copy methods of construction used in laying cast-iron pipe when laying the more brittle sewer pipe.

Concrete Cradles.—The development of the cradle of concrete used at Washington to carry the pipe sewers is shown in Fig. 110. The 1871–1879 section had a mortar joint and vitrified band and the pipes were without sockets, which has been true of all pipes used down to the present time. When Lieut.-Col. Lansing H. Beach was in charge of the sewerage work there, he reported that "the bottom of the sewer, with this pipe, can be made much more even and free from projections due to irregularities of circumference" than with socket pipe. The first section was probably laid many years prior to 1871, according to information furnished by A. E. Phillips, former superintendent of the sewer department of the District of Columbia, but that date is the beginning of the publicly permitted use of sewers of this type for sanitary drainage. From 1879 to 1888 the pipe rested in a cradle of natural cement concrete 22 in. wide on the bottom and 6 in. thick under the pipe, while the joint

was made with a vitrified band and a ring of mortar 4 in. thick, 14 in. wide at the pipe, and 6 in. wide on top. The 1888–1894 cradle was widened to 24 in. but otherwise it and the joint were unchanged. The 1894–1903 cradle remained unchanged but the collar was left

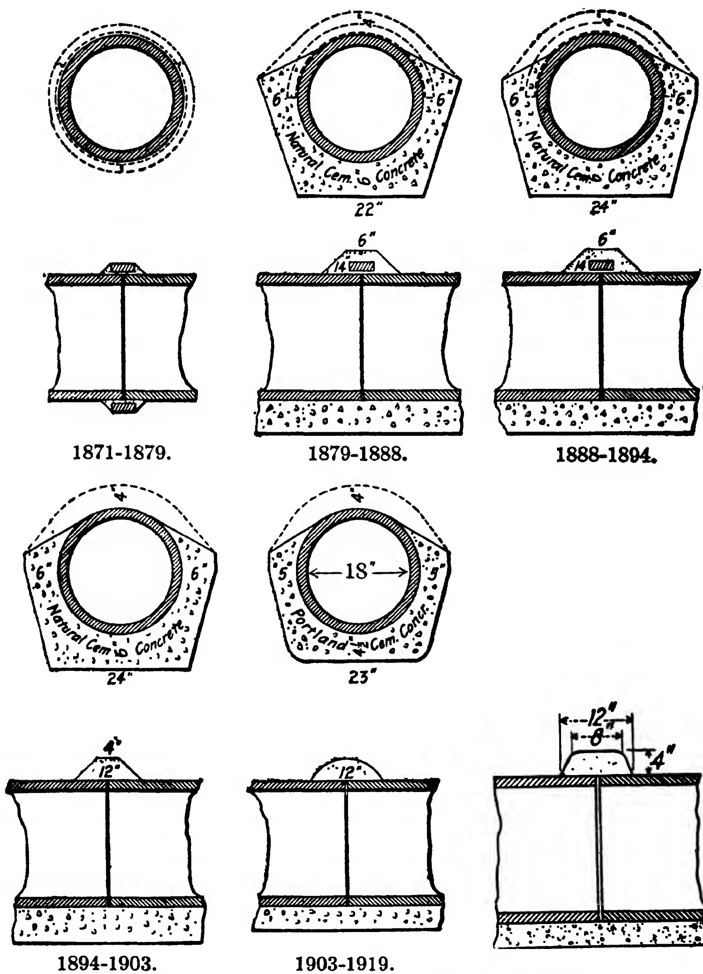


FIG. 110.—Cradle and joint of Washington pipe sewers.

out of the joint. The 1903–1914 cradle was made of portland cement concrete and its dimensions were reduced a little, and the joint was given an entirely new cross-section. The concrete envelope was first adopted in 1879, according to Mr. Phillips, as a preventive of root

intrusion, by Captain Hoxie while engineer commissioner of the District.

Figure 111 shows three different types of concrete cradles used with socket and spigot pipe.

Figure 112 shows a recent design (1925) of concrete cradle for a 24-in. pipe sewer at Louisville, Ky. This design is credited to W. M. Caye,

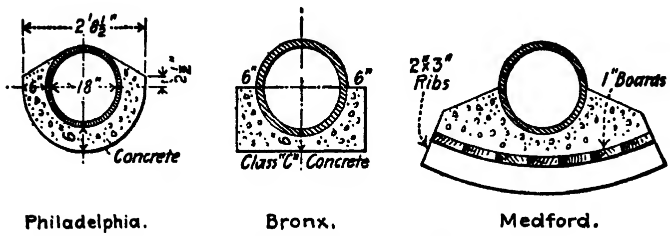


Fig. 111.—Types of cradles.

designing engineer for the Commissioners of Sewerage of Louisville. It has been found that pipe sewers of vitrified clay or concrete, without the concrete cradle and at the depths at which these sewers are usually laid in that city, have cracked and failed. In the Louisville practice

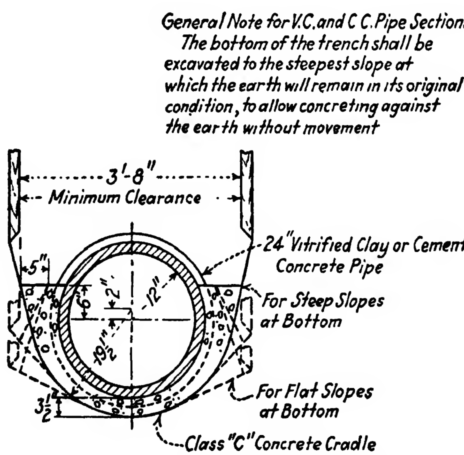


Fig. 112.—Concrete cradle for 24-in. pipe sewer, Louisville, Ky.

the cradle is paid for by the linear foot. Concrete is not allowed to be deposited against the sheeting, so it is especially to the advantage of the contractor not to drive the sheeting any deeper than is necessary. The forms of the cradle corresponding to these different conditions of earth are shown, varying with the depth to which the sheeting must be driven.

CHAPTER XII

MASONRY SEWERS

Definition.—For the purpose of this discussion, a masonry sewer is considered to be one built in place in the trench, whether the material be brick, terra-cotta block, monolithic concrete, or a combination of materials.

Notation.—The special notations used in this chapter are as follows:

B = outside diameter of pipe
D = inside vertical diameter
d = inside diameter of circular section
H = height of backfill over top of pipe
R = radius of arch
S = clear span of arch
r = rise of intrados
t = floor or bottom thickness at sidewall
t_c = thickness of arch at crown
t_s = thickness of arch at springing line
V = weight of backfill per square foot
W = inside horizontal diameter or width
W_1 = uniform live load per square foot

Other notations are employed on some of the illustrations, but their meanings are clearly shown.

Types of Cross-section.—The majority of the masonry sewers constructed in this country have been of circular cross-section, although in some old systems many sewers constructed with an oval or egg-shaped section are to be found. Since about 1900 a number of other sections have come into use and some of them have found quite general favor. In the following paragraphs, the principal types are described and some of their chief advantages and disadvantages discussed. The names given to the sections are usually descriptive of the form of the arch or upper part of the section, but are sometimes inaccurate.

Circular Section.—The circular section encloses a given area with the least perimeter and on that account affords the greatest velocity when half full or full. Under ordinary conditions, circular sections are economical in the amount of masonry required, although in flat-bottomed trenches or under conditions requiring special foundations, such as piles or timber platforms, additional masonry is required to support the arch. In the combined system, where the dry-weather flow of

sewage is very small in comparison to the storm-water flow, the velocity for the low flows is comparatively small in the circular section and on that account this section may not be as advantageous, theoretically, as the egg-shaped section.

For sewers under 5 ft. in size the circular form is usually employed in preference to other types.

Egg-shaped Section.—In combined sewers where the dry-weather flow of sewage is small compared with the capacity of the sewer required for storm water, or in separate sewers for a district where the present population is but a small proportion of the ultimate development, the ideal sewer section is one in which the hydraulic radius remains constant as the depth of flow decreases. It is impracticable to obtain the ideal, but the egg-shaped or oval section, theoretically, comes nearer to it than any other thus far devised.

In some cases the attempt has been made to design an egg-shaped section to meet special conditions, such as limited head room, or to proportion the radii of the oval to provide for special variations between the normal and maximum flows. This has led to some forms which have found little favor in this country, although used extensively abroad.

The standard egg-shaped section shown in *a*, Fig. 113, was designed in England by John Phillips about 1846 and has been used considerably since that date without modification. He also designed a "new" egg-shaped section, shown in *b*, Fig. 113, for use where the normal flow is extremely small compared with the maximum, but this has not been used extensively. The advantage of the egg-shaped sewer is that for small flows the depth is greater and the velocity somewhat higher than in a circular sewer of equivalent capacity. The depth of flow in the egg-shaped sewer is always greater than in the circular sewer for equal quantities, and for the small flows this increase in depth produces better flotation for the solid matter and consequently better actual velocity than if the floating solids should become stranded, thus causing obstructions to the flow.

The egg-shaped section has the disadvantages of being less stable, more liable to crack, requiring more masonry, and being more difficult to construct. In very stiff soil or in rock, it is sometimes possible to excavate the bottom of the trench to conform to the shape of the invert of the sewer, but, in general, in yielding soil or where foundations are poor, requiring piles or timber platforms, the egg-shaped section requires considerable masonry backing below the haunches to support the arch, even more than in the case of the circular sewer. For this reason, the egg-shaped section will be found in most cases more expensive than the circular type and, in the larger sizes, far more expensive than some of the other types which are discussed further on.

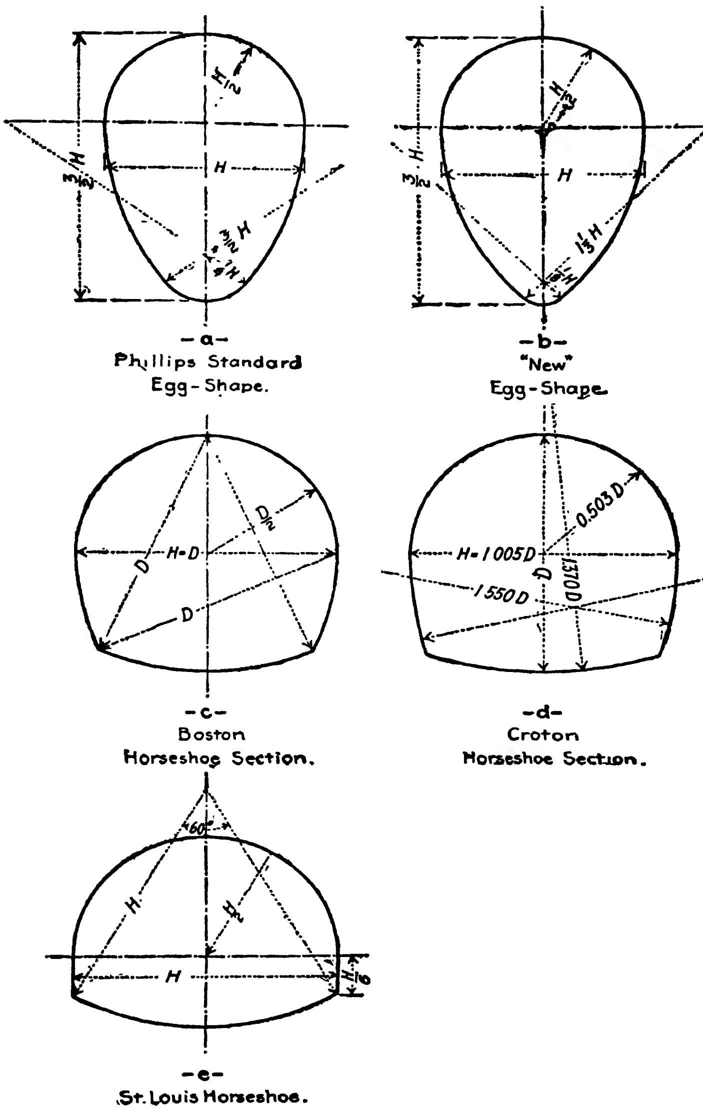


FIG. 113.—Typical cross-sections of sewers.

Catenary Section.—This section was used extensively on the Massachusetts North Metropolitan sewerage system, under the direction of Howard A. Carson. Its principal advantage is in the fact that it conforms so nearly in shape to the available space inside the wooden timbering in earth tunnels, as may be seen in Fig. 124, p. 425. The section is strong in that the line of resistance is well within the arch section. It has fairly good hydraulic properties, and the center of gravity of the wetted area is much lower with respect to the height than in the case of the circular section. This last fact may be of some advantage in locating lateral connections at a lower elevation, or in raising the invert of the main sewer. The former, of course, contemplates the possible operation of the lateral sewers under a head at times when the main sewer is running full. There are cases where this may be practical, but, in general, it should be avoided. It is of material advantage, however, where the allowable difference in water level is small. A larger quantity can be carried for a given increase in depth than is the case with the circular sewer. The catenary section has been but little used of late years.

Gothic Section.—This section, closely resembling the circular in shape and advantages, was also used to some extent on the North Metropolitan sewerage system in Massachusetts (see Fig. 36, p. 130). The horizontal diameter is about 17 per cent less and the vertical diameter about 8 per cent more than the diameter of the equivalent circle, and on that account it requires less width of trench than the circular section. Its greater height may be disadvantageous except under special conditions, because of the increased quantity of excavation required when the sewers are designed with hydraulic gradient at the crown. As may be expected, the hydraulic properties are not far different from those of the circular section. The Gothic or pointed arch is somewhat stronger than the semicircular. This section is not in general use at the present time, although it has advantages for special cases.

Basket-handle Section.—This section (Fig. 126, p. 428) was developed by Carson on the Massachusetts North Metropolitan sewer work and has been used to a large extent on that system, and also to some extent in other places. It is so nearly a horseshoe type that it is hard to draw a definite line between the two. Concerning this standard section (Fig. 126*b*) Carson states in his Third Annual Report to the Metropolitan Sewerage Commission, for the year ending Sept. 30, 1891:

The horizontal diameter is about 6 per cent less than the vertical. The arch is slightly pointed and the invert is flatter than a semicircle. In this shape the area, perimeter, and the theoretical velocity, when flowing more than one-sixth full, differ but little from the corresponding elements in a circle having the same height. In actual construction, under the conditions that usually obtain on our work, this shape is more stable when entirely

completed, than a circular shape. It requires more care, however, to prevent injury to the invert, while the latter is being constructed.

In general the basket-handle section has about the same advantages and disadvantages as the horseshoe type, described next, into which it merges so that it is difficult to determine whether some sections should be classed as basket-handle or horseshoe. The invert with the large radius curve and rounded corners between the side walls and invert, may have some advantage in strength, but the difference is so slight and the difficulties of construction so much greater than the form usually employed in the horseshoe type that it is not now in general favor. There may also be some slight advantage in having the Gothic arch because of greater strength and somewhat greater ease in removing collapsible arch forms.

Horseshoe Section.—For large sewers it is probable that next to the circular section this has been more generally used than any other. Many horseshoe sections have been developed to meet varying conditions, a few of which are shown in Figs. 127, 128, and 129, pages 430, 431 and 433, respectively. Above the springing line the horseshoe section has a semicircular arch, while the side walls below the springing line are vertical or incline inward, sometimes in a straight line and sometimes curved. The form of the invert varies in section from a horizontal line to a circular or parabolic arc, or other design intended to concentrate the low flows near the center of the invert.

One great advantage of this type of sewer is that it conforms to the shape of the bottom of the trench as usually excavated and on that account does not require the large increase in masonry backing to sustain the arch, which is needed with circular and egg-shaped sewers built in anything but rock or the firmest of soil. Another advantage is that for a given width or horizontal diameter the sewer may be designed with less height as a horseshoe and still have the same carrying capacity as the circular section. Where the depth of the sewer below the surface is controlled by the grade at the crown, there would be a consequent saving in excavation because of the decrease in depth. Where only a restricted amount of head room is available, the wide horseshoe type can often be used to advantage. In yielding soil where it is necessary to spread the foundations, the horseshoe type can be used in an economical manner, because of the saving of masonry in the invert. The limit of the horseshoe section along this line is the semicircular section, in effect a horseshoe section without side walls.

The chief disadvantage of the horseshoe section is that, unless the side walls are made heavy, the stability of the arch must depend to some extent on the ability of the earth backfilling to resist the lateral thrust of the arch transmitted to the side walls near the springing line. The

effect of the side walls is to increase greatly the bending moment at the crown and center of invert, especially the latter. If the sewer is constructed of monolithic reinforced concrete, with continuous reinforcing bars from the center of the invert to the crown of the sewer, the reinforcement required to resist the bending moment at the crown and invert center and also at the springing line will be considerable.

If the horseshoe section is constructed in rock cut, so that the invert or base of the side walls can be built directly on the rock, the conditions will be greatly altered and the line of resistance will be within the section much more often than if the sewer is constructed in compressible soil, with the whole structure acting as an elastic ring from the center of the invert to the crown. Brick arches in compressible soil require the construction of comparatively heavy side walls or abutments. The use of reinforcing metal in concrete has helped to remedy this condition, but, even with heavily reinforced sections, cases have been known where the arch cracked on the inside at the crown and on the outside at the quarter points or down toward the springing line. While this did not produce failure, it was objectionable from the point of view of leakage and rusting of the steel reinforcing metal. It is impossible to conceive of the passive resistance of the earth, especially in newly backfilled trenches, being brought into action without some movement of the concrete to compact the particles of earth next to the masonry. The effect of such movement on the stability of brick arches is well illustrated in a paper by Alphonse Fteley[1] on "Stability of Brick Conduits."

Semielliptical Section.—The arch of this section is either a true semiellipse, or is made up of three or more circular arcs approximating the semiellipse. As the center of gravity of the wetted area is lower with respect to the crown than in the circular sewer, the normal flow line will be much lower, which, as mentioned in the discussion of the catenary section, may be of considerable advantage (see Fig. 130, p. 435).

The chief advantage of this type of sewer is that the shape of the arch more nearly coincides with the line of resistance under actual working conditions than is the case with other sections. Because of this, the arch section can be made relatively thin and still keep the stresses in the masonry within allowable limits. The section is dependent for stability on the lateral pressure of the earth to only a small extent. The fact that the arch is of thin section and goes nearly to the invert makes it more necessary to design and construct the invert so as to distribute the pressure over a sufficiently large area.

This section depends to a larger extent on the stability of the invert than is the case with the sections previously mentioned. Where a sewer of the semielliptical section is constructed in compressible soil and the structure is built monolithic, with reinforcing bars running continu-

[1] *Jour. Assoc. Eng. Soc.*, 1883; **2**, 123.

ously from the center of the invert to the crown of the sewer, there will be a large bending moment at the center of the invert. Under such foundation conditions, the invert should be made as thick as the arch at the springing line and should be heavily reinforced to withstand the stresses. Unless this is done, cracks are likely to occur at the center of the invert.

As in the horseshoe type, the invert of the semielliptical section readily conforms to the bottom of the trench excavation, and for that reason the quantity of masonry below the springing line is not excessive.

This section is not as advantageous for low flows as the circular, because of the wide and shallow invert in which there is a very low velocity. For sewers where the quantity to be carried is not subject to wide variations, however, and the normal flow is as much as one-third of the total capacity of the sewer, this disadvantage nearly disappears. The hydraulic properties of the semielliptical section are good in general, which, with the desirable structural features, make this type one of the best for sewers over 6 ft. in diameter.

Parabolic or Delta Section.—This type of sewer was designed and built by W. B. Fuller[1] at Duluth, Minn., in 1888 and later was used by James H. Fuertes[2] for the sewerage system of Santos, Brazil. In 1902 Fuertes designed a similar section for Harrisburg, Pa., shown in Fig. 133*a*, page 439.

The sewer section is nearly triangular in shape, the arch being a parabola and the invert a short circular arc joining tangents with slopes of about three horizontal to one vertical. The section shown has a somewhat larger carrying capacity than that of a circular section of the same height. It is both economical and strong, and has the added advantage that the normal flow line is lower than in the circular section. This is especially valuable in districts where the available fall is limited, as in cities where the effect of tide water requires the sewers to be built in shallow cut. The V-shaped invert is well adapted for low flows. In this section, as in the semielliptical type, the shape of the arch nearly coincides with the line of arch resistance, which results in a strong section. It has the disadvantage as compared with the semielliptical type, however, of requiring a wider space for equal capacity and height, because of the pointed arch. For locations where there is but little depth of excavation, as in crossing low land, the section has a further advantage because the wide space makes it possible to construct the foundation to better advantage, and the greater carrying capacity below the springing line makes it possible to build a section of less height than in the case of the circular sewer, and where the sewer has to be covered by an embankment this involves a smaller quantity of earth work.

[1] *Eng. News*, 1890; **24**, 374.
[2] *Eng. Record*, 1894; **29**, 252.

Elliptical Section.—A few sewers have been constructed in this country with a true elliptical section (Fig. 125, p. 427), some with the longer axis vertical and others with it horizontal. This section is unlike the semielliptical type in that both above and below the springing line, it is an approximation to a portion of an ellipse. Unless the excavation is made in very firm soil, there will be additional masonry backing required below the haunches to support the arch, and this is usually an objection from the point of view of economy in masonry. In general, this shape is difficult to construct, and because there are so few points to commend the section it has not come into general use.

U-shaped Section.—In cases where the width of trench is limited and sufficient head room is available to build a sewer whose vertical diameter is materially greater than the horizontal, the U-shaped section has some advantages (see Fig. 135*e* and *f*, p. 442). The hydraulic properties of the section of Fig. 135*e* are fairly good until it becomes filled, when the hydraulic mean radius is materially reduced due to the addition of the width of the roof to the wetted perimeter. The invert is well adapted to maintain good velocities for low flows. It also has the advantage, because of the pointed shape, of offering a little less difficulty to the withdrawal of forms than the circular invert. In proportion to its area it requires considerable masonry and on that account is not economical for large sewers, but for sewers in the vicinity of 3 ft. in size, it doubtless has advantages for special conditions.

Rectangular Section.—This type has been used for many years where the head room or side room in the trench was limited, but more recently the rectangular section has been used for main lines because of the simplified form work, easy construction, economy of space in the trench both as regards width and head room and economy of masonry, especially with light cover, and also because of its excellent hydraulic properties at all depths less than full. As may be seen from the diagram of hydraulic elements (Fig. 47, p. 136), the velocity and discharge are relatively large just before the flat top is wet, but decrease very much as soon as the wetted perimeter is increased and the hydraulic radius decreased by the wetting of the top. On this account it is customary in designing rectangular sections to allow an air space above the maximum flow line of from 3 to 12 in., depending upon the size of the sewer and the amount of head room available.

Where the trench is in deep rock cut, this type can be used to great advantage, as is pointed out by Horner.[1] With a narrow, high section, the width of excavation can be reduced materially, often more than enough to offset the increase in depth of trench required. The hydraulic properties of the section become less favorable as the ratio of the height to the width increases; Horner found the economical ratio to be between

[1] *Eng. News*, 1912; **68**, 426.

1.5 and 2. A section of this type is shown in Fig. 132*b*, p. 438. The more common form of rectangular section has a greater width than height, as may be seen in Fig. 134, p. 440. This section requires careful designing to insure its stability. The flat slab top if used must be designed as a beam to carry the earth load and the side walls must be strong enough to resist the lateral earth pressure. If the top is built in the form of a flat arch, the side walls must be strengthened to carry the thrust of the arch.

In some cases the flat top has been constructed with I-beams encased in concrete, but this method is not economical of steel as the I-beams are designed to carry the load while the concrete merely acts as a filler between the beams and as a protection to the steel. This method, however, has the advantage of making it possible to complete the sewer and backfill the trench more quickly than where the roof is a slab reinforced with bars. The steel beams can be placed very easily and quickly and do not require such constant inspection as is the case with slabs reinforced with bars. It is claimed that in some cases this ease of construction will offset the additional cost of the steel, and in many cases where a large sewer is built in a congested district, it is of considerable advantage to be able to backfill the trench with the least possible delay.

The V-shaped invert is frequently used with the rectangular section on account of its suitability for low flows.

Semicircular Section.—This type of sewer, examples of which are shown in Fig. 131, page 436, has been employed rather extensively in New York City and vicinity, and is particularly adapted to the use of brick or stone masonry. Its most frequent use has been for outfall sewers crossing low land, where the natural surface of the ground is largely below the top of the sewer arch and in places even below the invert. As with the other arched sections, the invert must be firm and well designed to support the thrust of the arch. Two of these sections are often built side by side as twin sewers, instead of one large sewer, in order to save head room.

As a rule, the section requires a larger amount of masonry in proportion to its capacity than other types. The hydraulic properties are not so advantageous as those of the rectangular section, which, since the advent of reinforced concrete, has commonly been used instead of the semicircular section. The semicircular section requires a wider trench and more extensive foundations for equal capacity and height than most of the other forms.

Sections with Cunette.—Various types of sewers have been constructed with a special dry-weather channel or cunette in the invert. This type, although used extensively in France and Germany, has been employed but little in the United States. The most notable example is in the trunk sewers at Washington, D. C. (Fig. 135*c*, p. 442).

This section requires additional masonry in the invert and a greater depth of trench, but has the advantage of providing a good channel in which self-cleansing velocities may be maintained when the flow is small.

Double and Triple Sections.—Where outfall sewers are located in thickly settled districts and the available head room is seriously limited, it sometimes is of advantage to divide the section into two or more waterways side by side in one structure. In other cases, where a storm-water sewer is constructed above or below a large sanitary sewer, it may be more economical to build both waterways in one structure, one over the other. Representative types of such structures are shown in Figs. 137, 138, and 139 (pp. 444 to 446).

SELECTION OF TYPE OF SEWER

The selection of the type of sewer depends upon a number of conditions, all of which must be carefully considered and balanced in the choice of the best type to build. In general, that sewer is the best which, for the least cost per linear foot, will be easy to maintain in operation and will have the requisite stability to withstand the external and internal forces. In the following paragraphs a number of the principal items to be considered are enumerated in detail.

Hydraulic Properties.—Diagrams of the principal hydraulic elements of a large number of sewer sections have been given in Chap. III (Figs. 32 to 48, pp. 128 to 136).

Theoretically, the best cross-section for a sewer, from the standpoint of hydraulics, with a given slope, S, and carrying a uniform quantity of water per unit of time, is the semi-circle for an open channel and circle for a closed channel, both running full; because the hydraulic mean radius, R, has a greater value for these sections than for any other of equal area, and consequently the velocity of flow is greater.

This theoretical advantage is partly offset by the fact that the flow in sewers is not uniform but is constantly changing in depth, and, therefore, the minimum velocity is an important consideration. The circular section is not as advantageous as the egg shape for low flows.

In some cases, a small semicircular channel has been constructed in a V-shaped invert to carry the minimum flow. Assuming a certain minimum velocity is to be maintained with a given minimum quantity of sewage, the diameter of the small semicircular channel required for this flow can readily be computed. The V-shaped invert with circular arc at the junction has been used to advantage with the rectangular sewer section and also with some of the other types.

Where the normal flow is equal to one-third or more of the maximum flow, the circular type is the best for velocity and carrying capacity, but there are other considerations which usually affect the form of the sewer and may dictate some other type.

In comparing one section with another, it is important to study the relation between the depth of flow and the corresponding velocity and discharge. The diagrams in Chap. III give, for each of the principal types of conduits, the ratio of each of the three hydraulic elements, area, mean velocity and discharge, of the filled segment to that of the entire section, corresponding to any ratio of depth of flow to vertical diameter.

Construction and Available Space.—The method of construction of a sewer, whether in open cut or in tunnel, may have an important influence on the selection of the type. In tunnel work especially, it is desirable to have a section which will utilize to the best advantage all of the space inside the tunnel bracing. In earth tunnels, where the common form of timbering is used, the catenary or semielliptical sections conform readily to the available space. In rock tunnels, the circular or horse-shoe sections are likely to be more advantageous. If the sewer is built in open cut, the form of section will be influenced by its ability to carry the earth loads.

Where the excavation is in rock or firm soil, it is often possible to shape the bottom of the trench to conform to the shape of the invert of the sewer and thereby save considerable thickness of masonry in such types as the circular or egg-shaped sections. If the excavation is in soft material, where the bottom of the trench must necessarily be flat, or if the sewer is to be built on piles or a timber platform, considerable additional masonry will be required for the circular or egg-shaped sewers.

The amount of space available for a sewer may be exceedingly limited. Sometimes the head room is limited because of the proximity of the surface of the street to the grade of the sewer, sometimes the side room is limited because of adjacent structures, and again the available depth may be limited on account of tide water or other conditions which control the allowable depth to the hydraulic grade line. The rectangular section has proved one of the most useful for such conditions, although the horse-shoe section, with horizontal and vertical diameters adjusted to meet the conditions, has been used frequently. In a few cases, the full elliptical section has also been used in restricted places. Where the hydraulic grade line depth is limited, it is desirable to use a sewer section which will carry the maximum and minimum flows with the least variation in depth of flow. The catenary, parabolic, semielliptical, and rectangular sections are especially suitable for this purpose, as the center of gravity of the wetted area is comparatively low in contrast to the circular section. The semicircular section has also proved useful in this connection, although the rectangular section is being used in preference in the more recent work of this character.

Cost of Excavation and Materials.—The cost of excavation required by one type as compared with another should be carefully considered, for if the excavation is in earth in a deep trench, it will probably be cheapest

to use a narrow and deep section and thereby save considerable width of excavation, even though the depth of excavation be slightly increased. This will be especially true in a rock trench, where it may be found of advantage to use a narrow rectangular section having a height 1½ to 2 times the width. For a sewer built in very shallow cut, or practically on the surface of the ground, a wider section will be advantageous, because little additional cost is incurred by increasing width, whereas greater depth may increase materially the cost of excavation. Furthermore, the cost of an embankment over a wide section will generally be less, because of reduced height. The parabolic or delta section is especially useful for crossing low lands where the sewer is largely out of the ground and must be covered by an embankment. The semicircular section has also been used for this purpose, but has been superseded more recently by the rectangular section, having a width about 1½ times its height.

In former years, many sewers were constructed of stone, but in recent years other materials have proved less expensive and better adapted to this type of construction and very few sewers are now built of stone. The costs of labor and construction materials vary greatly in different localities and this may influence to a large extent the type of construction selected.

In general, from the structural viewpoint, a self-supporting sewer is more desirable than one which depends partially on the passive resistance of the earth backfilling for its stability. When comparing the relative costs of sewers of several types, care should be taken to see that the masonry sections are structurally comparable. For example, it is obviously unfair to compare a monolithic concrete sewer of the type shown in Fig. 115, page 406 with a two-ring brick sewer, unless account is also taken of their relative stabilities.

The object in designing a sewer section should be to obtain one in which the quantity of masonry and other materials is a minimum consistent with the requisite stability, hydraulic properties, and other considerations.

For sewers in which the normal flow is at least one-third of the maximum flow, it has been found that the semielliptical section is economical in masonry and at the same time provides for the other requirements.

Stability.—The structure must be designed to carry the load of earth on backfill above it as well as any superimposed load. The circular arch is not as strong as either the Gothic, the parabolic, or the semielliptical arch. The semicircular arch depends for stability to a great extent upon the lateral pressure of the sides of the trench, and also to a certain extent on the lateral resistance or passive pressure of the earth backfilling, although this can be obviated by increasing the thickness of the side walls or abutments. The semicircular sections obviate part of this difficulty by omitting the side walls and resting the arch directly on the

invert or foundation. In a rock trench the ability of the sides of the trench to resist pressure is so great that the side walls of the sewer can be greatly reduced in thickness, the thrust of the arch being carried directly into the rock. In this case a very flat arch can be used to advantage.

Imperviousness.—Where a sewer is to be constructed under a river bed or below the water table, it may be of particular importance for the walls of the sewer to be impervious. To this end, if the sewer is built of concrete, it is desirable to insert longitudinal reinforcing bars in the concrete with a total area of 0.2 to 0.4 per cent of the sectional area of the concrete, in order to distribute the stress throughout the length of the sewer barrel and thereby prevent the formation of cracks which would permit leakage.

While the possibility of leakage or infiltration does not ordinarily determine the shape of a sewer, it is worthy of consideration when the selection is to be made. For example, if a sewer is to be built below the water table it may be well to adopt a section which is least likely to crack, whereas under other conditions the advantages of a different section might be sufficiently great to warrant its use even though small arch cracks were to be expected.

SELECTION OF SIZE OF SEWER

In Chaps. V and VIII, methods are given by which the quantity of sewage and storm water for which the sewers are to be designed, can be estimated. In determining the size of sewer to carry this estimated quantity, an additional factor of safety is often allowed by computing the sewer as flowing less than completely full, as one-half or two-thirds full. Such an allowance does not seem to be logical, for uncertainties as to the quantity of sewage produced and the hourly, daily, and seasonal variations should be considered in estimating these quantities, the sewer being designed to carry them without further allowances, its capacity corresponding to the maximum estimated quantity of sewage.

Designs of rectangular and U-shaped sewers should be based upon the capacity when completely full, unless a very liberal air space is provided, since there are several factors not considered in design which might cause the water to rise to the roof if the space were small. As can be seen from Figs. 46 and 47, pages 135 and 136, both the velocity and discharge are materially reduced when the inside perimeter of the sewer becomes completely wet, owing to the reduction in the hydraulic mean radius.

Hydraulic Diagrams and Tables.—Diagrams giving the discharge of circular conduits can be used to compute the velocity and discharge in the case of conduits of other shapes, provided the hydraulic mean radius of the section in question is known. Any two sewers having the same

hydraulic mean radius and constructed on the same slope, will theoretically have the same velocity, but not necessarily the same discharge, owing to the difference in the area of the sections.

If we know the hydraulic mean radius of a special section, as, for example, a parabolic section, we can find the corresponding velocity from the diagram for circular conduits for any specified slope; and from the product of the velocity thus obtained by the area of the parabolic section, the corresponding discharge of that section can be computed.

Where considerable work is to be done with one type of sewer of different sizes, it will be found a great convenience to construct a diagram for it, in order to save computations. Such diagrams are given in Figs. 32 to 48 inclusive (Chap. III).

Data of this character may also be arranged in the form of a table, similar to Table 33, page 140, which gives the values of the hydraulic elements of the Boston type of horseshoe section, as computed by F. A. Lovejoy of the Boston Sewer Department. These values are based on Kutter's formula for $n = 0.013$. The Boston type of horseshoe section is shown in Fig. 113c, page 391. The values in the table multiplied by $\sqrt{S}$, will give the corresponding discharge of the sewer flowing full. The form of this table is that used by P. J. Flynn in "Hydraulic Tables" (Van Nostrand Science Series).

Equivalent Sections.—A diagram (Fig. 114) designed by Frank Allen and Otis F. Clapp[1] shows the dimensions of equivalent horseshoe and circular conduits flowing full, based on Kutter's formula with $n = 0.013$. The form of the horseshoe section is shown in the figure, D being the vertical diameter, W the horizontal diameter; the radius of the side walls, $2W$, and the radius of the invert $2W$. By equivalent conduits is meant conduits having equal carrying capacities but not necessarily equal areas. In this type of horseshoe section, the arch is always semicircular. The limiting cases covered by this diagram are a section having only arch and invert, in which D is $0.5635W$, and a section in which $D = W$.

The following modified form of Kutter's formula given in Swan and Horton's "Hydraulic Diagrams" was used in computing this diagram:

$$v = \left(\frac{Z}{1 + \frac{x}{\sqrt{R}}}\right)\sqrt{RS}$$

in which x and Z are empirical constants. The quantities x and Z vary but slightly between wide limits in the value of S, and may therefore be considered approximately constant within such limits. With $n = 0.013$ and S between the limits of 0.001 and 0.010, $x = 0.551$ and $Z = 181.69$, with sufficiently close approximation; while for S between 0.010 and 1.00,

[1] *Eng. Record*, 1904; **50**, 430.

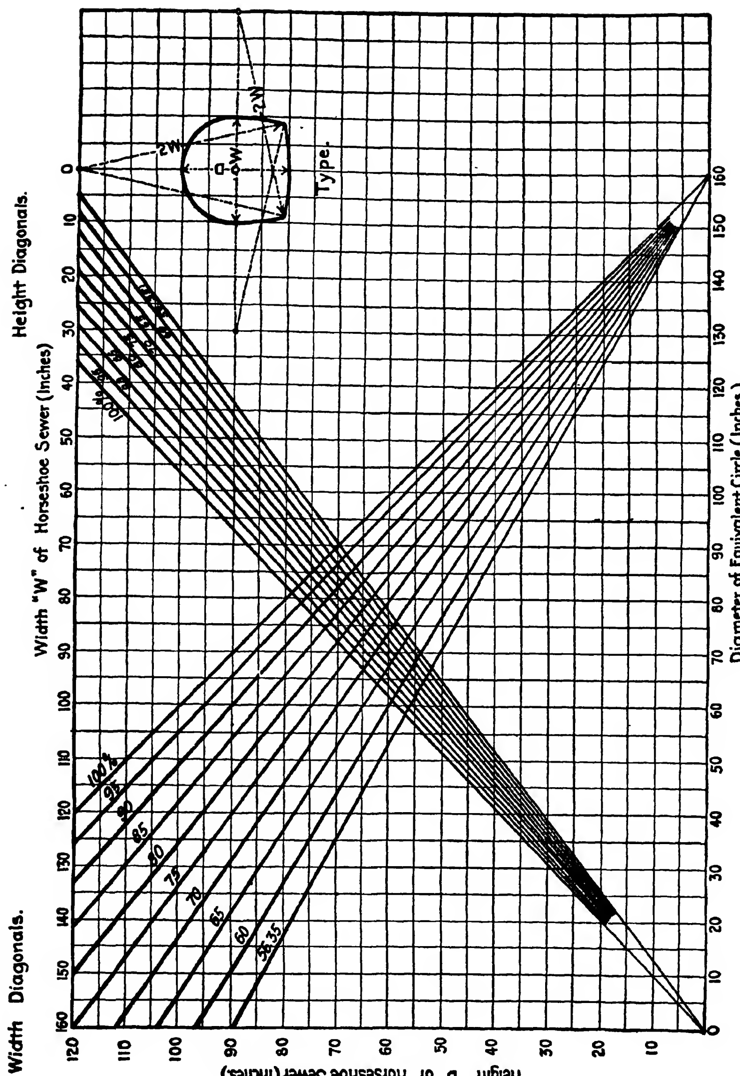


FIG. 114.—Diagram of equivalent horseshoe and circular conduits flowing full, with $n = 0.013$.

$x = 0.542$ and $Z = 181.02$. (This is, in effect, a still more "simplified" form of the "simplified Kutter formula" referred to in Chap. II, p. 93.)

In order to describe the method of using the diagram the following example is quoted:[1]

Required a horseshoe shape 78 in. high, equivalent in discharging capacity when flowing full to a 96-in. circular section. Find 78 at the left and 96 at the bottom of the diagram; trace the horizontal line through 78 to its intersection with the vertical through 96, which falls upon a height diagonal numbered 65; then trace along the 78 horizontal again, to the right or left, as the case may require, until the 65 width diagonal is met; then look to the top and find 120 for the width of the horseshoe. All dimensions are given in inches. A 78- by 120-in. section of the type shown is equivalent in flowing capacity to a 96-in. circle.

For sections larger than those plotted on the diagram, a convenient fraction, such as one-third, of the dimensions may be taken, and the results increased three times to obtain the desired figures.

DESIGN OF CROSS-SECTION

In selecting the dimensions of the masonry section to avoid excessive stresses in the masonry and at the same time be economical of material, it is unwise to reduce the thickness to theoretical limits on account of the uncertainty as to the quality of work obtainable. The saving by using extremely thin sections with high stresses is small and may prove to be false economy. For masonry sewers 5 ft. and less in size, the thickness of the best section will often depend more on the minimum thickness allowable on account of construction methods than on the stresses developed in the section. For plain and reinforced concrete sewers, a minimum crown thickness of 5 in. is considered good practice, but a less thickness is not desirable when the intention is to obtain first-class work. Some experienced engineers would adopt a minimum thickness considerably greater than 5 in. because of the uncertainty of securing satisfactory concrete under the conditions with which they have had to contend; they would arbitrarily increase the thickness of masonry throughout.

Empirical Formulas for Thickness of Arches.—In selecting the dimensions of a trial arch section, the following formulas may be of assistance. They should not be relied upon, however, to determine the final section. The formulas are only approximate and do not take into account many of the conditions which should govern the design of an arch.

F. F. Weld[2] gives the following:

The writer has devised the following equation, based upon a study of all available data upon the subject and his own experience in designing arches

[1] *Eng. Record*, 1904; **50**, 430.

[2] *Eng. Record*, 1905; **52**, 529.

for a great variety of conditions. He believes it a safe guide for all ordinary conditions of span and load:

$$t_c = \frac{1}{12}(\sqrt{S} + 0.1S + 0.005W_l + 0.0025V)$$

where W_l = live load uniformly distributed, and V = weight of earth fill over the crown, both in pounds per square foot. The arch ring at the quarter points should have a depth of from $1\frac{1}{4}t_c$ to $1\frac{1}{2}t_c$, depending upon the curves of the intrados.

Taylor and Thompson[1] state that the Weld formula gives fairly correct results in ordinary cases.

Obviously, the thickness for a hingeless arch should increase from the crown to the springing. The radial thickness of the ring at any section is frequently made equal to the thickness at the crown multiplied by the secant of the angle which the radial section makes with the vertical. For a three-centered intrados and an extrados formed by the arc of a circle, these trial curves may be at the quarter points a distance apart of $1\frac{1}{4}$ to $1\frac{1}{3}$ times the crown thickness and at the springings two to three times the crown thickness.

American Civil Engineers' Handbook[2] gives the following formulas for the approximate thickness of a masonry arch at the crown for spans under 20 ft.:

First-class ashlar.................... $t_c = 0.04(6 + S)$
Second-class ashlar or br ck............ $t_c = 0.06(6 + S)$
Plain concrete .... $t_c = 0.04(6 + S)$
Reinforced concrete.................. $t_c = 0.03(6 + S)$

The thickness of masonry at the springing line may be computed in the following manner from the crown thickness, as given by the above formulas.

Add 50 per cent for circular, parabolic, and catenarian arches having a ratio of rise to span less than $\frac{1}{4}$. Add 100 per cent for circular, parabolic, catenarian, and three-centered arches having a ratio of rise to span greater than $\frac{1}{4}$. Add 150 per cent for elliptical, five-centered, and seven-centered arches. These thicknesses should be measured along radial joints.

It is also stated that the crown thicknesses, computed by the above formulas, should be increased about 60 per cent for culverts under a high fill and about 25 per cent for railroad arches.

Frye[3] states that the following formulas give very close results for first-class concrete and cut-stone work:

For highway bridges,

$$t_c = \sqrt{0.01S\left(\frac{S}{r} + 3\right)} + 0.15$$

[1] "Concrete, Plain and Reinforced," Third Edition, 715.
[2] Fifth Edition, 968.
[3] "Civil Engineers Pocketbook," 1913; 766.

For high highway embankments or for railroad bridges,

$$t_c = \sqrt{0.01S\left(\frac{S}{r} + 4\right)} + 0.20$$

For high railroad embankments,

$$t_c = \sqrt{0.01S\left(\frac{S}{r} + 5\right)} + 0.25$$

For all cases

$$t_s = t_c[1 + 0.002(S + 2r)]$$

EXAMPLES OF SEWER SECTIONS

Sewer Sections Actually Used.—The designing engineer will derive much assistance from a study of sewer sections used by other engineers. The dimensions of many such sections are available, although so scattered through engineering literature as to make difficult a ready comparison of their salient features.

In cities where considerable sewer construction is in progress, it has often been found advantageous to formulate a set of standard sections for sewers of different sizes, thus making it unnecessary to prepare special designs for each sewer. These standard sections, especially for the smaller sizes, have been based largely on the analysis of a number of sections previously adopted, and upon experience in their construction. They are valuable, therefore, as representing the judgment and experience of engineers with respect to sewers actually constructed and as not necessarily being confined to theoretical lines.

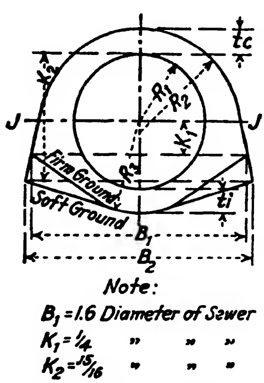


FIG. 115.—Louisville standard concrete section.

The data relating to and the illustrations of sewer sections presented in the following pages, should be considered merely as furnishing to the designing engineer suggestions which he may find helpful in preparing designs for the particular work in hand. As the local conditions attending the construction of these sewers cannot be accurately known, it should not be assumed that any of them can be adopted without modification for the conditions surrounding the work in hand.

Standard Sewer Sections.—In Figs. 115 to 119, inclusive, and in Tables 120 to 124 are shown a number of sections adopted as standards at various places.

Louisville, Ky.—The cross-sections of plain concrete sewers shown in Fig. 115 and Table 120, were prepared for the Commissioners of Sewerage of Louisville, Ky., J. B. F. Breed, Chief Engineer. The dimensions

given were based on what experience had shown to be a safe thickness of masonry under the conditions there existing. The minimum thickness at the crown and at the invert was fixed at 5 in. because of the practical difficulty of obtaining with certainty a first-class wall of monolithic concrete of less thickness. The shape of the masonry invert is dependent upon the character of the excavation, whether it is in firm ground or soft ground, these being the terms applied to materials which would and would not stand when trimmed to the shape of the firm ground section. For sewers of this type constructed on timber platforms or piles the line of the under side of the concrete invert should be horizontal. For reinforced-concrete sections, the thickness of masonry shown for the larger diameters may be somewhat reduced.

TABLE 120.—DIMENSIONS OF PLAIN CIRCULAR CONCRETE SEWERS, LOUISVILLE

Dimensions of the sections										Quantity of concrete, cu. yd. per lin. ft. sewer	
Diameter	t_s	t_c	$B_1/2$	$B_2/2$	K_1	K_2	R_1	R_2	R_3	Firm ground	Soft ground
24"	5"	5"	1' 7¼"	1' 8¼"	6"	1' 10½"	1' 0"	1' 6"	1' 5"	0.13	0.15
27"	5"	5"	1' 9⅜"	1' 10¾"	6¾"	2' 1 5/16"	1' 1½"	1' 8"	1' 6½"	0.15	0.18
30"	5"	5"	2' 0"	2' 1½"	7½"	2' 4⅛"	1' 3"	1' 10"	1' 8"	0.18	0.21
33"	5"	5"	2' 2⅜"	2' 4¼"	8¼"	2' 6 15/16"	1' 4½"	2' 0"	1' 9½"	0.19	0.23
36"	5"	5"	2' 4¾"	2' 6¾"	9"	2' 9¾"	1' 6"	2' 2"	1' 11"	0.22	0.26
39"	5"	5"	2' 7¼"	2' 9¼"	9¾"	3' 0 9/16"	1' 7½"	2' 4"	2' 0½"	0.25	0.29
42"	6"	6"	2' 9⅝"	3' 0"	10½"	3' 3⅜"	1' 9"	2' 6"	2' 3"	0.29	0.35
45"	6"	6"	3' 0"	3' 2½"	11¼"	3' 6 3/16"	1' 10½"	2' 8"	2' 4½"	0.33	0.40
48"	6"	6"	3' 2⅜"	3' 5¼"	1' 0"	3' 9"	2' 0"	2' 10"	2' 6"	0.38	0.45
51"	6"	6"	3' 4¾"	3' 8"	1' 0¾"	3' 11 13/16"	2' 1½"	3' 0"	2' 7½"	0.41	0.49
54"	6"	6"	3' 7¼"	3' 10¾"	1' 1½"	4' 2⅝"	2' 3"	3' 2"	2' 9"	0.43	0.53
57"	6"	6"	3' 9⅝"	4' 1½"	1' 2¼"	4' 5 7/16"	2' 4½"	3' 4"	2' 10½"	0.47	0.57
60"	6"	7"	4' 0"	4' 4"	1' 3"	4' 8¼"	2' 6"	3' 6"	3' 0"	0.53	0.65
63"	6"	7"	4' 2⅜"	4' 6½"	1' 3¾"	4' 11 1/16"	2' 7½"	3' 8"	3' 1½"	0.57	0.71
66"	6"	7"	4' 4¾"	4' 9¼"	1' 4½"	5' 1⅞"	2' 9"	3' 10"	3' 3"	0.61	0.77
69"	6"	8"	4' 7¼"	5' 0"	1' 5¼"	5' 4 11/16"	2' 10½"	4' 0"	3' 4½"	0.66	0.84
72"	6"	8"	4' 9⅝"	5' 2¾"	1' 6"	5' 7½"	3' 0"	4' 2"	3' 6"	0.70	0.88

Plane surfaces may be used for the exterior of the sewer and this may occasionally be more economical[1] although involving additional masonry.

Borough of the Bronx.—Figure 116[2] shows the standard forms of circular and egg-shaped sewers, constructed of unreinforced concrete. The minimum thickness of masonry, as given in these tables, is 6 in. for a minimum diameter of 33 in.

[1] *Eng. News-Record*, 1920; **85**, 148.

[2] In "Standard Details of Construction," 1913, Borough of the Bronx, N. Y., Richard H. Gillespie, Chief Eng. of Sewers and Highways. This section is still (1928) standard.

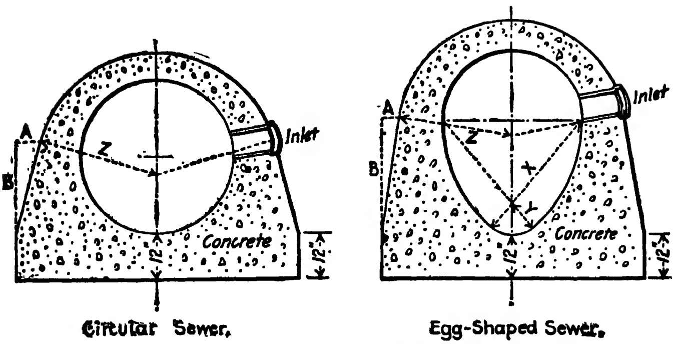


FIG. 116.—Standard plain concrete sections. (*Bronx.*)

TABLE 121.—STANDARD PLAIN-CONCRETE SECTIONS, BOROUGH OF THE BRONX, NEW YORK CITY

Circular	Crown	Width of base	Outside radius Z	Offset A	Offset B	Concrete area, square feet
2′ 9″	6″	5′ 3″	2′ 1½″	7⅜	1′ 9¹¹⁄₁₆″	11.94
3′ 0″	6″	5′ 6″	2′ 3″	7¼″	1′ 11¹⁄₁₆″	12.82
3′ 3″	8″	6′ 3″	2′ 7½″	7⁵⁄₁₆″	2′ 0½″	16.41
3′ 6″	8″	6′ 6″	2′ 9	7³⁄₁₆″	2′ 1⅞″	17.46
3′ 9″	8″	6′ 9″	2′ 10½″	7⅛″	2′ 3¼″	18.52
4′ 0″	8″	7′ 0″	3′ 0″	7¹⁄₁₆″	2′ 4⅝″	19.60

Egg shaped	Crown	Width of base	Outside radius Z	Radius X	Radius Y	Offset A	Offset B	Concrete area, sq. ft.
29″ × 40″	6″	4′ 9″	1′ 11½″	2′ 10⅛″	7½″	5⁷⁄₁₆″	2′ 3⅛″	12.82
32″ × 44″	6″	5′ 0″	2′ 1″	3′ 0⁷⁄₁₆″	7½″	5⁷⁄₁₆″	2′ 5½″	14.00
34″ × 46″	6″	5′ 3″	2′ 2″	3′ 2″	9″	6″	2′ 6¹⁵⁄₁₆″	14.78
38″ × 50″	8″	6′ 0″	2′ 7″	3′ 2³⁄₁₆″	9″	5⁷⁄₁₆″	2′ 8³⁄₁₆″	19.08
40″ × 53″	8″	6′ 3″	2′ 8″	3′ 4¹¹⁄₁₆″	9″	6″	2′ 10⁷⁄₁₆″	20.33
42″ × 56″	8″	6′ 6″	2′ 9″	3′ 9⅞″	12″	6½″	3′ 0¾″	21.43

Gregory's Semielliptical Section.—A standard semielliptical section, shown in Fig. 117, was worked out in 1910 by John H. Gregory in connection with the preparation of plans for a large trunk sewerage project. He stated that this section, which was designed to be built of concrete, is better adapted for sewers 6 ft. and over in size than for smaller ones. The several dimensions are given in terms of the height, *D*. The functions of the diameter were so chosen, that with increments of 3 in. in the height, the resulting dimensions will come out in whole inches or inches and fractions of an inch in common use, as, for example, quarters,

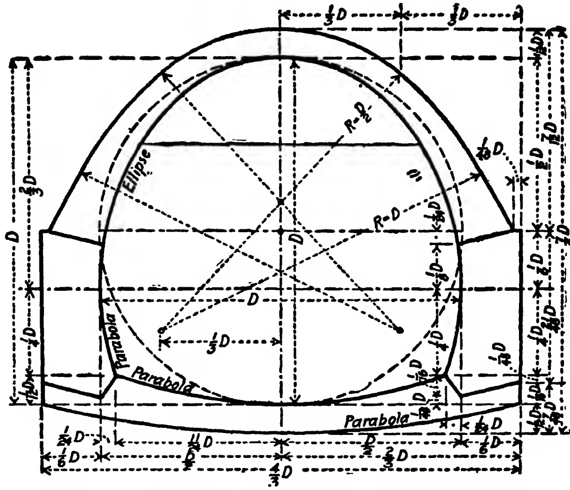


FIG. 117.—Gregory's standard semi-elliptical section.

TABLE 122.—AREA AND VOLUME OF MASONRY IN SEMIELLIPTICAL SEWERS

Gregory's Section (Fig. 117)

Inside diameter of sewer D (1)	Area, square feet			Volume of masonry, cubic yards per linear foot $0.01657D^2$ (5)
	Gross area on outside lines $1.265D^2$ (2)	Area of section inside $0.8176D^2$ (3)	Net area of masonry $0.4475D^2$ (4)	
6′ 0″	45.54	29.43	16.11	0.597
6′ 6″	53.45	34.54	18.91	0.700
7′ 0″	61.99	40.06	21.93	0.812
7′ 6″	71.16	45.99	25.17	0.932
8′ 0″	80.97	52.33	28.64	1.061
8′ 6″	91.40	59.07	32.33	1.197
9′ 0″	102.5	66.23	36.25	1.342
9′ 6″	114.2	73.79	40.39	1.496
10′ 0″	126.5	81.76	44.75	1.657
10′ 6″	139.5	90.14	49.34	1.827
11′ 0″	153.1	98.93	54.15	2.005
11′ 6″	167.3	108.1	59.18	2.192
12′ 0″	182.2	117.7	64.44	2.387
12′ 6″	197.7	127.7	69.92	2.590
13′ 0″	213.8	138.1	75.63	2.801
13′ 6″	230.6	149.0	81.56	3.021

eighths, or sixteenths. Gregory further stated that the section is suitable for use only where the conditions are such that the side walls will be firmly supported by the sides of the trench. Where these conditions cannot be obtained, the side-wall sections should be modified to meet the conditions.

The horizontal and vertical diameters of the section are the same, and the horizontal diameter is located one-third the height above the bottom of the sewer. The gross area of this section on outside lines equals $1.2651D^2$, the area of the section inside equals $0.8176D^2$, and the net area of masonry equals $0.4475D^2$.

Table 122 shows the area of these sections and the net volume of masonry in cubic yards per linear foot for each size from 6 ft. to 13 ft. 6 in. in diameter by 6-in. steps. Additional data in regard to the hydraulic elements of this section are given in Table 32 and Fig. 44, while the velocity and discharge for various diameters are shown in Fig. 20, following page 90. Gregory further stated[1]

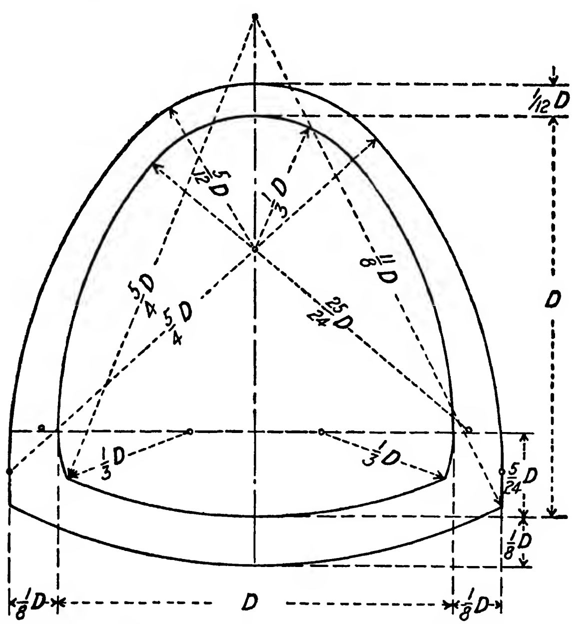


FIG. 118.—Metcalf and Eddy's standard semielliptical section.

In conclusion it should be pointed out that the dimensions given for the masonry section are a *minimum* and that not only would the best of materials and workmanship be required, but also careful inspection. Where

[1] *Eng. News*, 1914; **71**, 552.

these conditions cannot be obtained or where the sewers would be required to carry heavy loads, the sections should be reinforced with steel or the dimensions increased, especially the arch and side walls.

TABLE 123.—MINIMUM DIMENSIONS OF AUTHORS' SEMIELLIPTICAL SEWER SECTION
See Fig. 118

1	2	3	4	5	6	7	8	9	10	11	12
			Thickness of concrete		Interior radii			Exterior radii			
Inside vertical diameter D ft. in.	Area of water-way sq. ft.	Hy-draulic mean radius ft.	Crown ft. in.	Center of in-vert and spring line ft. in.	Crown intra-dos and side wall ft. in.	Side intra-dos ft. in.	Invert and side extra-dos ft. in.	Crown extra-dos ft. in.	Invert ft. in.	Area of con-crete, sq. ft.	Quan-tity of con-crete cu. yd. per lin. ft.
6 0	28 2	1 442	0 6	0 9	2 0	6 3	7 6	2 6	8 3	14 12	0 523
6 6	33 1	1 562	0 6½	0 9¾	2 2	6 9¼	8 1½	2 8½	8 11¼	16.58	0 614
7 0	38 4	1 683	0 7	0 10½	2 4	7 3½	8 9	2 11	9 7½	19 21	0 712
7 6	44 05	1 803	0 7½	0 11¼	2 6	7 9¾	9 4½	3 1½	10 3¾	22 08	0 817
8 0	50 1	1 923	0 8	1 0	2 8	8 4	10 0	3 4	11 0	25 10	0 930
8 6	56 6	2 043	0 8½	1 0¾	2 10	8 10¼	10 7½	3 6½	11 8¼	28.35	1 054
9 0	63 4	2 163	0 9	1 1½	3 0	9 4½	11 3	3 9	12 4½	31.80	1 177
9 6	70 7	2 284	0 9½	1 2¼	3 2	9 10¾	11 10½	3 11½	13 0¾	35 41	1.311
10 0	78 3	2 404	0 10	1 3	3 4	10 5	12 6	4 2	13 9	39 24	1 453
10 6	86 3	2 525	0 10½	1 3¾	3 6	10 11¼	13 1½	4 4½	14 5¼	43 26	1 602
11 0	94 75	2 646	0 11	1 4½	3 8	11 5½	13 9	4 7	15 1½	47 48	1 757
11 6	103.5	2 764	0 11½	1 5¼	3 10	11 11¾	14 4½	4 9½	15 9¾	51.89	1.921
12 0	112.75	2 884	1 0	1 6	4 0	12 6	15 0	5 0	16 6	56 51	2.092
12 6	122 4	3 005	1 0½	1 6¾	4 2	13 0¼	15 7½	5 2½	17 2¼	61.31	2.270
13 0	132 4	3 125	1 1	1 7½	4 4	13 6½	16 3	5 5	17 10½	66.32	2 456
13 6	142 7	3 245	1 1½	1 8¼	4 6	14 0¾	16 10½	5 7½	18 6¾	71 51	2.649
14 0	153 5	3 365	1 2	1 9	4 8	14 7	17 6	5 10	19 3	76 91	2.849

Area of waterway = $0.7831D^2$. Area of concrete section = $0.3924D^2$.

Authors' Semielliptical Section.—The details of the semielliptical section shown in Fig. 118 and Table 123 were developed by the authors from the experience in constructing sewers of this type at Louisville, Ky. In all of the principal types the stresses were carefully analyzed, but no definite standards were developed in the Louisville work, and on that account the sections actually constructed vary slightly from the one shown. This sewer is intended to be constructed of concrete reinforced with steel bars. The dimensions given are probably the least which should be used for the sizes tabulated; and it is probable that, for smaller sizes, the thicknesses at crown and invert should not be less than those for the 6-ft. sewer, namely, 6 and 9 in., respectively.

For any given conditions, the stresses which would exist should be determined by the methods described in Chap. XIV, and the thickness of concrete and amount of steel required to sustain these stresses should be provided.

The Dresser formulas on p. 517 are applicable to the semielliptical section shown in Fig. 118 and Table 123. By their solution, the required thickness of concrete and amount of steel can be approximated with a minimum of labor and with sufficient accuracy for nearly all purposes.

St. Louis Five-centered Arch.—The standard cross-section of the five-centered arch or semielliptical type of sewer shown in Fig. 119 was

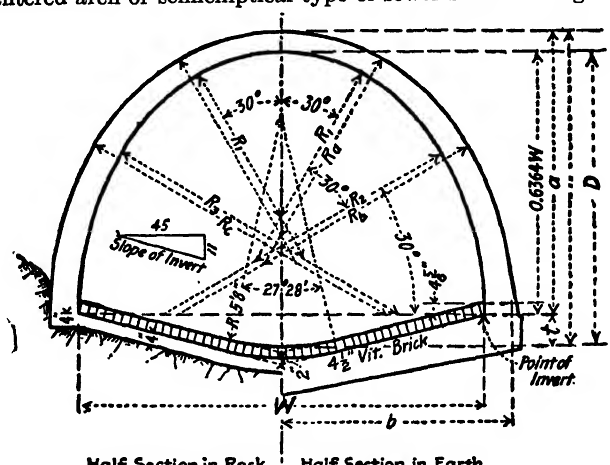


FIG. 119.—St. Louis five-centered arch sewer.

$R_1 = 0.4048W$; $R_2 = 0.5285W$; $R_3 = 0.7774W$; $R_a = 2.7321b - 0.8458(a + b)$; $R_b = 0.4650(a + b)$; $R_c = 1.8862(a + b) - 2.7321b$; $t = 0.1222W - 0.1667$; $D = 0.7568W - 0.5521$; Area $= 0.5609W^2 - 0.3854W - 0.1509$; Wetted perimeter $= 2.8211W - 0.8239$.

furnished by W. W. Horner. Table 124 gives the leading dimensions and hydraulic properties of this section. The following notes in regard to the design of this standard section have been taken from a paper by P. J. Markmann, Office Engineer, St. Louis Sewer Department.

In the preliminary studies, three systems of external forces were studied. The first, called the "standard" system, was composed of vertical forces due to the total weight of the backfill resting on the sewer arch and a small amount of horizontal earth pressure, depending in amount upon the angle of repose of the earth, assumed to be 25 deg. The second system of external forces consisted of vertical forces only and ignored the existence of any horizontal earth pressure. This case would express the condition of the angle of repose approaching 90 deg., and would cover the possible case of horizontal forces in the "standard"

system of loading, having been assumed too great as compared with the vertical forces. The third system consisted of external forces acting normal to the center line of the arch, these forces being assumed equal to the weight of the fill, which is equivalent to a very wet condition or hydrostatic pressure. In each case, analyses were made for varying depths of fill, 10 ft., 20 ft., 30 ft., and 40 ft. from the ground surface to the crown of the sewer.

TABLE 124.—PROPERTIES OF FIVE-CENTERED ARCH SEWER (FIG. 119)

W ft.	R_1, ft. in.	R_2, ft. in.	R_3, ft. in.	t, ft. in.	D, ft. in.	Area sq. ft.	Wetted perim. ft.	Hyd. rad. ft.
6	2 5$\frac{1}{8}$	3 2$\frac{1}{16}$	4 8	0 6$\frac{13}{16}$	4 0	17.729	16.103	1.101
7	2 10	3 8$\frac{3}{8}$	5 5$\frac{5}{16}$	0 8$\frac{1}{4}$	4 9$\frac{1}{8}$	24.635	18.924	1.302
8	3 2$\frac{7}{8}$	4 2$\frac{3}{4}$	6 2$\frac{5}{8}$	0 9$\frac{3}{4}$	5 6$\frac{3}{16}$	32.664	21.745	1.502
9	3 7$\frac{3}{4}$	4 9$\frac{1}{16}$	6 11$\frac{15}{16}$	0 11$\frac{3}{16}$	6 3$\frac{5}{16}$	41.813	24.566	1.702
10	4 0$\frac{9}{16}$	5 3$\frac{7}{16}$	7 9$\frac{5}{16}$	1 0$\frac{11}{16}$	7 0$\frac{7}{16}$	52.085	27.387	1.902
11	4 5$\frac{7}{16}$	5 9$\frac{3}{4}$	8 6$\frac{5}{8}$	1 2$\frac{1}{8}$	7 9$\frac{1}{2}$	63.479	30.208	2.101
12	4 10$\frac{5}{16}$	6 4$\frac{1}{8}$	9 3$\frac{15}{16}$	1 3$\frac{5}{8}$	8 6$\frac{5}{8}$	75.994	33.029	2.301
13	5 3$\frac{1}{8}$	6 10$\frac{7}{16}$	10 1$\frac{1}{4}$	1 5$\frac{1}{16}$	9 3$\frac{3}{4}$	89.631	35.850	2.502
14	5 8	7 4$\frac{13}{16}$	10 10$\frac{5}{8}$	1 6$\frac{9}{16}$	10 0$\frac{13}{16}$	104.390	38.671	2.699
15	6 0$\frac{7}{8}$	7 11$\frac{1}{8}$	11 7$\frac{15}{16}$	1 8	10 9$\frac{15}{16}$	120.271	41.492	2.899
16	6 5$\frac{3}{4}$	8 5$\frac{1}{2}$	12 5$\frac{1}{4}$	1 9$\frac{7}{16}$	11 7	137.273	44.314	3.098
17	6 10$\frac{9}{16}$	8 11$\frac{13}{16}$	13 2$\frac{9}{16}$	1 10$\frac{15}{16}$	12 4$\frac{1}{8}$	155.397	47.135	3.297
18	7 3$\frac{7}{16}$	9 6$\frac{3}{16}$	13 11$\frac{15}{16}$	2 0$\frac{7}{8}$	13 1$\frac{1}{4}$	174 644	49.956	3.496
19	7 8$\frac{5}{16}$	10 0$\frac{1}{2}$	14 9$\frac{1}{4}$	2 1$\frac{7}{8}$	13 10$\frac{5}{16}$	195.011	52.777	3.695
20	8 1$\frac{1}{8}$	10 6$\frac{7}{8}$	15 6$\frac{9}{16}$	2 3$\frac{5}{16}$	14 7$\frac{7}{16}$	216.501	55.599	3.894
22	8 10$\frac{7}{8}$	11 7$\frac{9}{16}$	17 1$\frac{1}{4}$	2 6$\frac{1}{8}$	16 1$\frac{5}{8}$	262 845	61.240	4.292
24	9 8$\frac{9}{16}$	12 8$\frac{1}{4}$	18 7$\frac{7}{8}$	2 9$\frac{3}{16}$	17 7$\frac{7}{8}$	313.678	66.883	4.690
26	10 6$\frac{5}{16}$	13 8$\frac{15}{16}$	20 2$\frac{9}{16}$	3 0$\frac{1}{8}$	19 2$\frac{1}{16}$	368 996	72.525	5.088
28	11 4	14 9$\frac{5}{8}$	21 9$\frac{3}{16}$	3 3$\frac{1}{16}$	20 8$\frac{1}{4}$	428.804	78.167	5.486
30	12 1$\frac{3}{4}$	15 10$\frac{5}{16}$	23 3$\frac{7}{8}$	3 6	22 2$\frac{1}{2}$	493.098	83.810	5.884

The line of pressure in the arch for the standard system of forces was found to be a close approximation to an elliptical curve, and as the forces were assumed symmetrical, the major axis of this ellipse coincided with the vertical axis of the arch.

The arches were actually designed with a curvature following that of the line of pressure of the standard system of forces. The line of pressure for the second system of forces fell inside the standard line, thereby causing negative bending moments between the crown and springing line in the arch. The line of pressure for the third system of forces, for nearly all depths of fill, fell outside the standard line of pressure, causing positive bending moments between the crown and springing line.

The sewer arch of any required size was designed of such varying thickness (increasing from crown to abutment) as to resist, in addition to the direct thrust, not less than 50 per cent of the moments indicated by the positions of the lines of pressure for each of the two extreme conditions of loading.

The hydraulic radius of this conduit is equal to the hydraulic radius of a circle whose diameter $d = 0.7422W$, where W is the horizontal diameter of the conduit. The area of the conduit is equal to the area of a circle whose diameter $d = 0.8W$. The hydraulic radius of the conduit is nearly 93 per cent of that of a circle of equal area.

Louisville Sewers.—During 1907 to 1913, inclusive, there were constructed at Louisville, Ky., the main and intercepting sewers of a comprehensive system of sewerage. On this work J. H. Kimball was Designing Engineer, J. B. F. Breed, Chief Engineer, and Harrison P. Eddy, Consulting Engineer. Practically all sewers were constructed of concrete, the majority of them being reinforced with steel bars. The sizes varied from 8 in. to 15 ft. Table 125 gives the principal dimensions of a number of the larger sewers and is of interest in connection with Fig. 118, as showing the thicknesses of masonry actually constructed at Louisville. Additional data concerning these sewers will be found in other chapters of this book.

TABLE 125.—PRINCIPAL DIMENSIONS OF SEWERS CONSTRUCTED IN LOUISVILLE, KY., 1907–1913

1	2	3	4	5	6	7	8
Interior dimensions			Thickness of concrete				
					At springing line		
Vertical diameter, ft. in.	Horizontal diameter, ft in.	Type of sewer	Crown, inches	Invert, inches	Arch, in.	Side wall, in.	Depth of fill over crown, ft.
15 2	15 6	Horseshoe	11	12	17	17	25
13 11	14 3	Horseshoe	8	10	12½	12½	15
13 8	14 0	Horseshoe	10	12	15	18⅝	30
12 9	13 0	Horseshoe	8	10	14½	14½	20
12 3	12 3	Semielliptical	9	9	14	14	25
10 0	10 0	Semielliptical	8	8	16	16	42
10 1½	10 7	Horseshoe	10	14	15	17¾	37
9 0	12 0	Horseshoe	8	8	17	17	15.5
9 0	9 0	Horseshoe	8	8	15	15	15
7 6	10 0	Horseshoe	8	8	12	12	13
8 3	8 3	Semielliptical	8	8	12	12	12
8 0	8 0	Semielliptical	9	7	15	15	15.5
7 0	7 0	Semielliptical	8	7	13	13	13
6 3	6 3	Semielliptical	6	6	8¾	8¾	10
5 6½	5 10	Horseshoe	6	6	7½	7½	17
5 6	5 6	Circular	6½	6	10	10	12
4 11	5 2	Horseshoe	7	6	10	10	18
4 6	4 6	Semielliptical	7	6	9	9	25
5 6	5 6	Circular	6	7	10	10	5
3 3	3 3	Circular	5	8	8	8	10

Data from contract drawings—Commissioners of Sewerage.

TABLE 126.—CONCRETE SEWER ARCHES IN EARTH, ST. LOUIS

Horizontal diameter, feet	Type	Depth of fill over crown, feet	Thickness of concrete			Materials per linear foot of sewer			Where used
			Crown, inches	Springing line, inches	Invert, inches	Cubic yards concrete	Cubic yards vit. brick invert lining	Pounds steel	
33	Ellipt.	20	12	25	48	6.048	0.471	551.23	A
33	Ellipt.	30	18	29	63	6.201	0.471	591.00	A
33	Ellipt.	40	22	31	72	7.433	0.471	916.00	A
28	Ellipt.	10	9	18	28	3.409	0.398	320.00	A
28	Ellipt.	20	12	18	39	4.025	0.398	383.00	A
26	Ellipt	10	9	16	25	3.162	0.370	313.00	A
26	Ellipt.	20	12	16	35	3.507	0.370	346.00	A
24½	Ellipt.	10	9	18	27	3.564	0.348	238.00	A
24½	Ellipt.	20	11	22	38	4.122	0.348	330.00	A
24½	Ellipt.	30	13⅝	24	48	4.558	0.348	332.00	A
23	Ellipt.	.....	13	....	32	5.534	0.328	349.80	C
22½	Ellipt.	10	8	15	22	2.872	0.319	266.50	A
22½	Ellipt.	20	9	20	34	3.428	0.319	329.00	B
22½	Ellipt.	30	10	24	44	3.917	0.319	293.00	B
22	Horse-s.	10	11	18	25	3.649	0.320	437.50	B
22	Horse-s.	15	13	22	30	4 440	0.320	509.00	B
20	Horse-s.	15	12	21	28	3 811	0.291	447.00	B
20	Horse-s.	20	14	24	32	4.444	0.291	468.50	B
18	Horse-s.	15	11	19	26	3.130	0.262	369.00	B
18	Ellipt.	....	12	.....	30	3.815	0.257	262.60	C
16	Horse-s.	10	9	15	20	2.189	0.233	307.00	B
16	Horse-s.	10	12	18	23	2.990	0.233	125.00	D
16	Horse-s.	20	15	24	30	3.810	0 233	185.50	D
16	Ellipt.	......	11	26	31	2.947	0 223	199.00	E
15½	Ellipt.	......	10	24	27	2 579	0.223	193.00	E
15½	Horse-s.	10	13	16	21	2.670	0.226	220.00	F
15½	Horse-s.	15	14	18	24	2.929	0.226	220.00	F
15	Ellipt.	10	7	......	14	1.561	0 214	117.00	E
15	Ellipt.	20	9	......	22¾	2.129	0 214	164.00	E
15	Ellipt.	30	10½	.....	30½	2 662	0.214	218.50	E
14	Horse-s.	10	12	18	23	2.617	0.203	134.00	D
14	Horse-s.	20	16	23	30	3.402	0.203	171.00	D
14	Horse-s.	10	10	15	22	2 093	0.203	236.50	B
13	Horse-s.	10	11	14	18	1.951	0 189	151 50	F
13	Horse-s	15	12	15	20	2.116	0.189	192.50	F
13	Horse-s.	20	13	18	24	2 399	0.189	216.00	F
13	Ellipt.	10	7	......	12½	1.229	0.185	99.50	E
13	Ellipt.	20	8½	.....	19½	1.593	0.185	136.50	E
13	Ellipt.	20	8	15	21½	1.671	0.182	113.13	A
12	Ellipt.	10	7	......	11¾	1.065	0.171	100.50	E
12	Ellipt.	20	8	15½	18½	1.462	0.171	111.50	E
12	Horse-s.	10	10	16	20	1.920	0 174	110.00	D
12	Horse-s.	20	14	21	27	2.620	0.174	140.00	D
11	Horse-s.	10	10	14	18	1.630	0.160	87.00	D
11	Horse-s.	20	14	18	21	2.150	0.160	129.50	D
11	Horse-s.	10	9	12	16	1.408	0.160	123.50	F
11	Horse-s.	15	10	14	18	1.585	0.160	123.50	F

TABLE 126.—CONCRETE SEWER ARCHES IN EARTH, ST. LOUIS.—(*Continued*)

Horizontal diameter feet	Type	Depth of fill over crown feet	Thickness of concrete			Materials per linear foot of sewer			Where used
			Crown, inch	Springing line, inch	Invert, inch	Cubic yards concrete	Cubic yards vit. brick invert lining	Pounds steel	
10	Horse-s.	15	9	12	17	1.297	0.145	131.50	F
10	Horse-s.	20	10	14	19	1.469	0.145	146.50	F
10	Horse-s.	10	10	14	18	1.507	0 145	74.50	D
10	Horse-s.	20	12	18	23	1.870	0.145	116.50	D

NOTE: Locations indicated by letters in last column:
A. River Des Peres, Tunnel Line
B. River Des Peres, River Line
C. Baden Public, first Section
D. South Harlem Joint
E. Glaise Creek Joint
F. Rock Creek Joint

St. Louis Sewers.—A considerable number of sewer sections of large size have been designed and constructed by the St. Louis Sewer Department, and data upon these sections are given in Tables 126 to 128. Two classes of concrete were used; mortar for Class *A* concrete had a ratio of 1 bbl. of cement to 7.6 cu. ft. of sand; and that for Class *B* concrete, 1 bbl. of cement to 11.4 cu. ft. of sand. The concrete was made by mixing with the broken stone or gravel an amount of mortar of the proper class 10 per cent in excess of the voids in the stone or gravel. For Class *A* concrete, the unit allowable stress in the concrete was assumed between 500 and 560 lb. per square inch; and that for Class *B* concrete from 400 to 450 lb.

These arches were designed independently for particular conditions of live load, vibration, and other conditions. The Baden sewer arch is of Class *A*; and the invert, of Class *B* concrete. All other elliptical sections are of Class *A* concrete throughout. The River Des Peres horseshoe sections are of Class *A* concrete throughout, while all the other horseshoe sections are of Class *B* concrete throughout.

TABLE 127.—CONCRETE SEWER ARCHES, ROCK BELOW POINT OF INVERT, ST. LOUIS

Horizontal diameter, in ft.	Type	Depth of fill over crown ft.	Thickness of concrete			Materials per lin. ft. of sewer			Where used
			Crown, in.	Springing line, in.	Invert, in.	Cu. yd. concrete	Cu. yd. vit. brick invert lining	Pounds steel	
33	Ellipt.	20	12	25	25	4.339	0.471	405.00	A
33	Ellipt.	30	18	29	29	5.136	0.471	450.00	A
33	Ellipt.	40	22	31	31	6.186	0.471	607.00	A
28	Ellipt.	10	9	18	18	2.357	0.398	243.00	A
28	Ellipt.	20	12	18	18	2.643	0.398	243.00	A
26	Ellipt.	10	9	16	16	2.029	0.370	258.50	A
26	Ellipt.	20	12	16	16	2.256	0.370	258.50	A
24½	Ellipt.	20	11	22	22	2.814	0.348	241.50	A
24½	Ellipt.	30	13⅝	24	24	3.285	0.348	217.21	A
23	Ellipt.	......	13	....	22	2.785	0.328	216.00	C
22½	Ellipt.	10	8	....	15	1.926	0.319	200.00	A
22½	Ellipt.	20	9	....	20	2.374	0.319	257.50	A
22½	Ellipt.	30	10	....	24	2.771	0.319	185.50	A
22	Horse-s	10	11	18	25	2.970	0.320	200.00	B
22	Horse-s.	15	13	22	30	3.367	0.320	238.50	B
20	Horse-s.	15	12	21	28	2.925	0.291	197.00	B
20	Horse-s.	20	14	24	32	3.250	0.291	201.00	B
18	Horse-s.	15	11	19	26	2.437	0.233	120.50	B
18	Ellipt.	....	12	....	20	2.036	0.257	150.00	C
16	Horse-s.	10	9	15	20	1.833	0.233	120.50	B
16	Horse-s.	10	12	18	23	2.160	0.233	102.50	D
16	Horse-s.	20	15	24	30	2.720	0.233	143.50	D
15½	Horse-s.	10	13	16	21	2.041	0.226	141.50	F
15½	Horse-s.	15	14	18	24	2.222	0.226	141.50	F
14	Horse-s.	10	10	15	22	1.622	0.203	84.50	B
14	Horse-s.	10	12	18	23	1.952	0.203	112.50	D
14	Horse-s.	20	16	23	30	2.478	0.203	139.50	D
13	Horse-s.	10	11	14	18	1.505	0.189	94.50	F
13	Horse-s.	15	12	15	20	1.608	0.189	118.00	F
13	Horse-s.	20	13	18	24	1.891	0.189	134.20	F
13	Ellipt.	20	8	15	15	1.167	0.182	88.50	A
12	Horse-s.	10	10	16	20	1.542	0.174	89.50	D
12	Horse-s.	20	14	21	27	2.599	0.174	114.00	D
11	Horse-s.	10	10	14	18	1.360	0.160	71.00	D
11	Horse-s.	20	14	18	21	1.680	0.160	106.00	D
11	Horse-s.	10	9	12	16	1.099	0.160	80.00	F
11	Horse-s.	15	10	14	18	1.224	0.160	80.00	F
10	Horse-s.	15	9	12	17	1.017	0.145	77.50	F
10	Horse-s.	20	10	14	19	1.139	0.145	85.00	F
10	Horse-s.	10	10	14	18	1.137	0.145	64.00	D
10	Horse-s.	20	12	18	23	1.350	0.145	92.50	D

NOTE: Locations indicated by letters in last column:

A. River Des Peres, Tunnel Line
B. River Des Peres, River Line
C. Baden public, first section
D. South Harlem Joint
E. Glaise Creek Joint
F. Rock Creek Joint

TABLE 128.—CONCRETE SEWER ARCHES, ROCK ABOVE POINT OF INVERT ST. LOUIS

Horizontal diameter in ft.	Type	Depth of fill over crown, ft.	Thickness of concrete			Materials per lin. ft. of sewer			Where used
			Crown, in.	Springing line in.	Invert, in.	Cu. yd. concrete	Cu. yd. vit. brick invert lining	Pounds steel	
22	Horse-s.	10	11	18	18	2.782	0.320	200.00	B
22	Horse-s.	15	13	22	22	3.160	0.320	238.50	B
16	Horse-s.	10	12	18	18	1.991	0.230	100.50	D
16	Horse-s.	20	15	24	24	2.809	0.230	140.50	D
15½	Horse-s.	10	13	16	16	1.939	0.226	141.50	F
15½	Horse-s.	15	14	18	18	2.100	0.226	141.50	F
14½	Horse-s.	10	12	15	15	1.727	0.211	129.50	F
14	Horse-s.	10	12	18	18	1.663	0.203	80.50	D
14	Horse-s.	20	16	23	23	2.075	0.203	137.00	D
13	Horse-s.	10	11	14	14	1.435	0.189	94.50	F
13	Horse-s.	15	12	15	15	1.521	0.189	118.00	F
13	Horse-s.	20	13	18	18	1.777	0.189	134.00	F
12	Horse-s.	10	10	16	16	1.286	0.174	89.00	D
11	Horse-s.	15	10	14	14	1.164	0.160	80.50	F
11	Horse-s.	10	9	12	12	1.039	0.160	80 50	F
10	Horse-s.	15	9	12	12	0.945	0.145	77.50	F
10	Horse-s.	20	10	14	14	1.067	0.145	89.00	F

NOTE: Locations indicated by letters in last column:
A. River Des Peres, Tunnel Line
B. River Des Peres, River Line
C. Baden Public, first section
D. South Harlem Joint
E. Glaise Creek Joint
F. Rock Creek Joint

Typical Sewer Sections.—A number of sewer sections are reproduced in Figs. 120 to 140 inclusive which are typical of different classes of structures designed to meet special conditions. All information is from official sources unless otherwise stated.

Figure 120*a*.—Massachusetts Metropolitan Sewerage Com'n, Neponset Valley Sewer, 1897, William M. Brown, Jr., Chief Engineer, Gothic section, 4 ft. 3 in. by 4 ft. 4¾ in. Depth of cover approximately 18 ft. Material excavated was sand, gravel, and clay.

Figure 120*b*.—Massachusetts Metropolitan Sewerage Com'n, Neponset Valley Sewer, 1897, William M. Brown, Jr., Chief Engineer, Gothic section, 4 ft. by 4 ft. 1½ in. Left half of figure, construction for rock tunnel; right half, construction for tunnel in hard gravelly soil.

Figure 120*c*.—Philadelphia, Pa., 1906, George S. Webster, Chief Engineer, standard circular sewer, 4 ft. 9 in. The right half shows minimum section; left half, construction in "reduced" cradle. Steel reinforcing over piles. Piles, 12-in. yellow pine 3 ft. apart both ways.

Figure 120*d*.—Philadelphia, Pa., standard sewer section, 1906, George S. Webster, Chief Engineer, circular sewer, 4 ft. 9 in. Right half of section, construction in "maximum cradle," on piles 3 ft. 6 in. c. to c. transversely, and 3 ft. c. to c. longitudinally. Steel reinforcing over piles. Left half of section, construction on platform and piles. Platform of 6-in. yellow pine planking on 8- by 8-in. yellow pine stringers 3 ft. apart longitudinally. Piles 12-in. yellow pine, 3 ft. apart longitudinally and 3 ft. 9 in. c. to c. transversely.

Figure 120*e*.—Truro, Nova Scotia, 1902, Lea & Coffin, Engineers, 27-in. circular sewer, monolithic concrete to springing line of brick arch. Concrete used because of cheapness under given conditions as compared with brickwork. *Eng. Record*, 1902; **46**, 136.

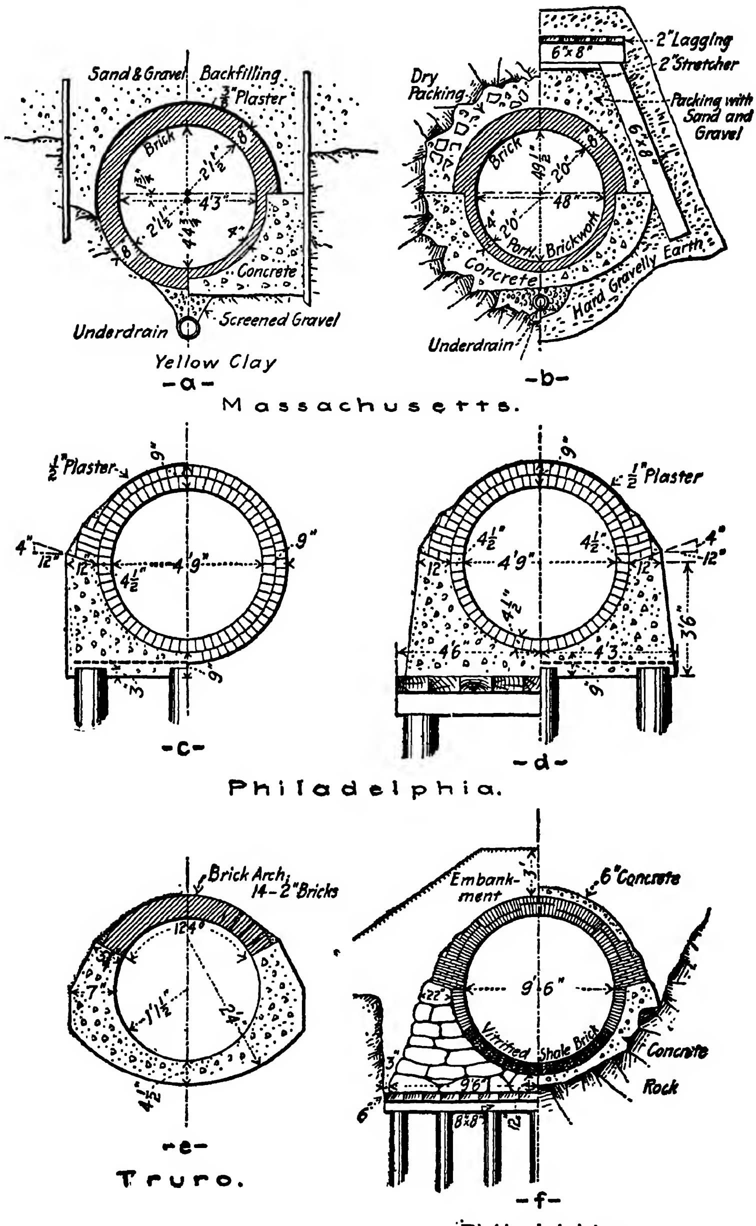


FIG. 120.—Typical circular sections with brick arches.

Figure 120*f*.—Philadelphia, Pa., Magee St. sewer, 1900, Geo. S. Webster, Chief Engineer, circular, 9 ft. 6 in. Left half of section, construction in earth cut on piles with earth cover. Platform yellow pine, 6-in. planks on 8- by 8-in. caps with 12-in. piles set 3 ft. c. to c. in each direction. Right half of the section, construction in rock cut.

Figure 121*a*.—Borough of Brooklyn, New York City, Gold St. Relief Sewer, 1907, E. J. Fort, Chief Engineer, circular section, 13 ft. 6 in. The figure shows two methods of construction. In a third method, the section was built entirely of concrete with 16 in. thickness at the crown and 20 in. at the springing line. A fourth type had a segmental concrete arch and concrete foundation of the same general dimensions as the left half of the section shown. Platform on earth, 2-in. plank laid on 4-in. sills; platform on piles, constructed of 6-in. plank floor laid on 10- by 12-in. capping on 12-in. spruce or pine piles, spaced 3 ft. 9 in. c. to c.

Figure 121*b*.—Borough of Queens, New York City. Trunk Sewer in Myrtle and St. Nicholas Aves., 1907, J. H. Johnson, Chief Engineer, circular section, 15 ft. Depth of cover about 15 ft.; excavation in dry sandy soil. Reinforced with Johnson corrugated bars "new style." Sections under 15 ft. of the same general form; 11-ft. 3-in. section 12 in. thick at crown and 27 in. at springing line. Arches in all sections less than 4½ ft. in diameter are 6 in. thick at the crown and 6 to 9 in. at springing line. Arch rings for sewers from 4½ to 5½ ft. in diameter are 6 to 8 in. thick at crown and 12 to 15 in. at the springing line. The 9½- and 10-ft. sections were similar to those just described. The thickness at crown and springing line of 9½ and 10-ft. sections was 12 and 24 in., respectively, for both sizes. At one point, cover over 9½-ft. sewer 22 ft. deep; sizes from 5½ ft. down had 8 to 10 ft. of cover. Practically no water encountered. *Eng. Record*, 1907; **101**, 599.

Figure 121*c*.—Des Moines, Iowa, Ingersoll Run Sewer, 1905, John W. Budd, City Engineer, 7-ft. circular sewer. *Eng. Record*, 1906; **58**, 537.

Figure 121*d*.—Toronto, Can., High-level intercepting sewer, 1910, Charles H. Rust, City Engineer. Circular reinforced concrete sewer on concrete piers crossing filled ground. Lining below springing line, vitrified brick. In trench, section was plain concrete with vitrified brick invert lining. Thickness at crown was 12 in.; at springing line, 17½ in.; at invert, 12 in.; invert below brick lining, 7½ in.; maximum width of plain concrete section 11 ft. 8 in.; concrete invert has horizontal base 3 ft. 6 in. wide and its sides slope upward 2 ft. 5 in. vertically in a horizontal distance of 4 ft. 1 in. *Eng. Record*, 1911; **63**, 301.

Figure 121*e*.—Wilmington, Del. Price's Run Sewer, 1903, T. Chalkley Hatton, Consulting Engineer, circular section, 6 ft. Left half for shallow cut where sewer was largely above ground; used with and without platform. Right half, construction entirely below ground. With a thickness of only 5 in. at the crown, the sections withstood without fracture all load they will be subjected to at any time. Reinforcement, woven wire fabric of No. 8 wire with No. 6 wire selvage and 6- by 4-in. mesh. A 6½-ft. sewer of same type with same thicknesses was constructed, but a 9-ft. 3-in. section had a crown thickness of 8 in., 12 in. at the springing line, and 8 in. of concrete at invert in section like right half of figure. Several hundred feet of this 9-ft. 3-in. section were built on pine piles 36 to 38 ft. long, four piles to each bent, spaced 3 ft. 10½ in. centers, and bents 4 ft. between centers. Each bent had a 10- by 12-in. yellow-pine cap carrying floor of 3- by 12-in. hemlock. Reinforcement, expanded steel, 6-in. mesh, No. 6 gage, approximately 2 in. from the inner surface. *Eng. Record*, 1904; **49**, 636.

Figure 121*f*.—Lancaster, Pa., 1903, Samuel M. Gray, Engineer, 6-ft. 10-in. circular sewer reinforced with 3-in. No. 10 expanded metal and inside below springing line lined with hard burned or vitrified brick. Alternative design had concrete foundation and brick arch; greater roughness estimated to require 2 in. more diameter, giving 38.48 sq. ft. as against 36.67 sq. ft. for concrete sewer; had three rings brickwork on concrete base 9 ft. wide, 6 in. thick below brick lining of invert and extending vertically on sides to springing line. Alternate section required 16.24 cu. ft. brick and 16.48 cu. ft. concrete per linear foot; quantities for concrete sewer illustrated were 13.79 cu. ft. concrete and 4.08 cu. ft. brickwork. Sewer constructed as illustrated on account of the greater comparative economy.

Figure 122*a*.—Massachusetts Metropolitan Sewerage Com'n, North Metropolitan System, 1893, Howard A. Carson, Chief Engineer. Gothic section, 8 ft. 10 in. by 9 ft. 4½ in.; built in pneumatic tunnel in soft clay underlaid by very wet sand. As an indication of the extent of groundwater, at one point it was impossible even with five compressors running to excavate nearer than 1½ ft. to grade of bottom of masonry until following method was used: Work started as low as possible, and concrete lining used for sides, roof, and heading. Then 2-in. tongued and grooved planks, set radially, prevented wet sand flowing in at the bottom.

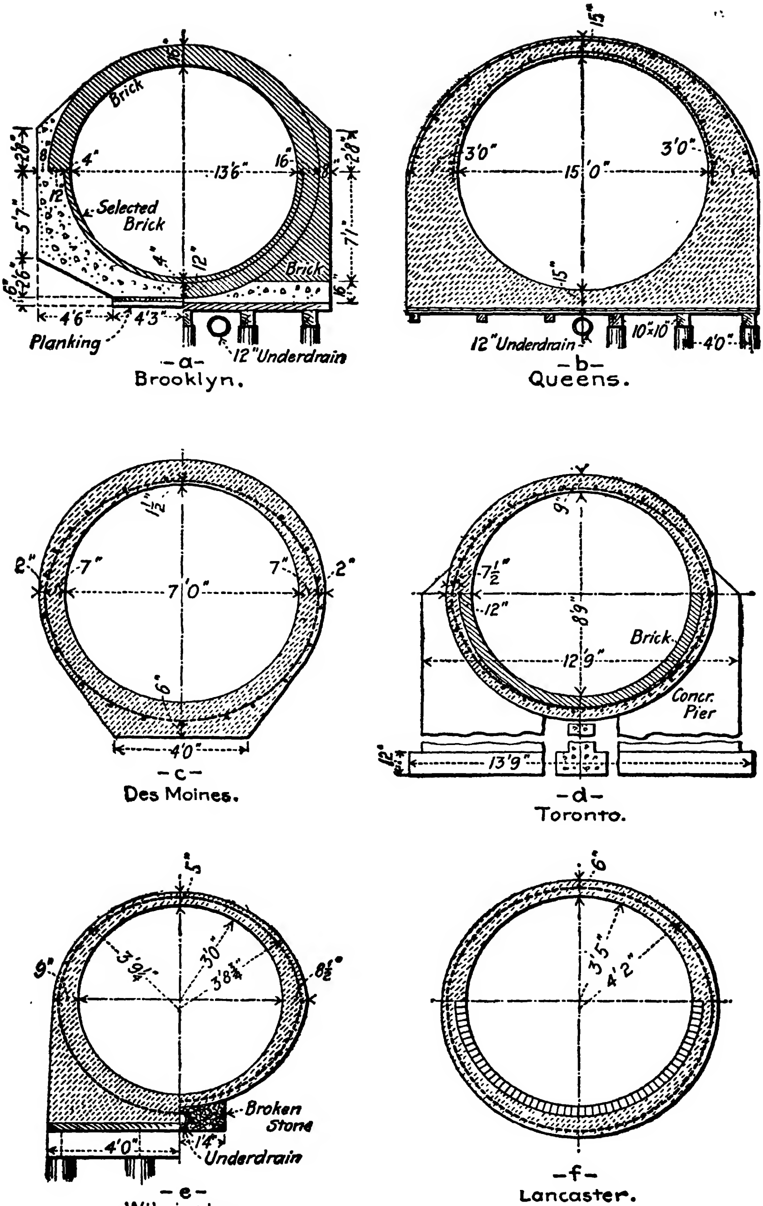


FIG. 121.—Typical circular sections.

Upper part of arch secured by ⅛-in. by 1-ft. by 3-ft. curved steel plates, bolted to each other and supported by 8- by 8-in. temporary posts. With concrete lining in place, it was possible to hold air pressure and allow remainder of excavation to be made and brick invert and linings set. Sides and bottom of section held in place by 2-in. plank lagging. Found later that with arch built first, same results were obtainable without use of concrete; brick arch built on wall plates, these and arch supported by braces from axial beam; invert then built up to wall plates, and the space left by removal of wall plates filled with brick. Brick arches always 12 in. thick. *Eng. News*, 1894; **31**, 121.

Figure 122*b*.—Cleveland, Ohio, Walworth Sewer, 1898, 10 ft. 3 in. circular section, very heavy on account of yielding plastic blue clay, unable to carry more than 2 tons per square foot. Thickness of arch increased gradually from crown to springing line and arch bricks arranged in alternate headers and stretchers in Flemish bond. To avoid excessively thick mortar joints, masonry was broken up as shown. Entire arch cut into segments separated

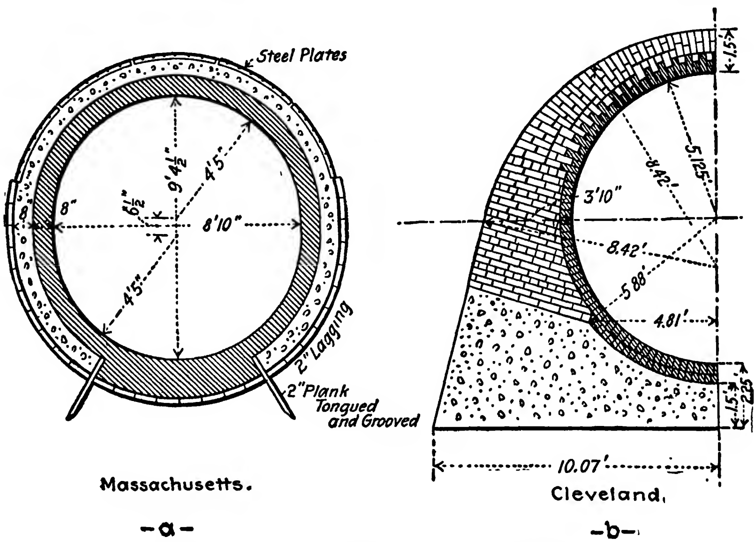


FIG. 122.—Typical gothic and circular sections.

by cylindrical surfaces and radial planes. Inner and outer faces of brick parallel with inner surface of completed sewer. Number of courses to build any particular cylindrical segment one more than the number in next inner segment and one less than number in next outer segment; mortar joints of ordinary thickness were thus obtained in all portions of arch. Surfaces separating inner and outer rings of segments, as well as extrados of arch, plastered with portland-cement mortar. Radial thickness of each part of superimposed masonry segments adjusted to break joints in adjoining segments by at least 4 in.

Sewer built on 3-in. oak plank, laid across sewer line on 3- by 12-in. oak sleepers, not more than 4 ft. c. to c. bedded in clay. Entire lower portion natural cement concrete. Top of concrete brought to plane inclining downward and inward four horizontal to one vertical. Minimum thickness of concrete under two rings of lining brick of the invert, 1.5 ft. for sewers from 8 ft. to 14 ft. 9 in. inclusive, and 2 ft. for larger sizes. Side walls brick laid in English bond in natural cement mortar, carried upward from concrete with courses pitching inward parallel to the upper surface of the concrete. Two concentric rings of brickwork in invert lining instead of one, in order to obtain a smoother inner surface.

Crown thickness fixed arbitrarily according to conditions in each case and thickness at springing line determined by formula $t_s = \dfrac{S}{\dfrac{S}{14} + 2.572}$ Thickness at any other point of

TABLE 129.—PRINCIPAL DIMENSIONS OF WALWORTH SEWER, CLEVELAND, OHIO

Diameter, ft. in.	Thickness of masonry in feet				Diameter, ft. in.	Thickness of masonry in feet			
	Crown	On horiz. line through center of sewer	Center of invert	Width of concrete foundation		Crown	On horiz. line through center of sewer	Center of invert	Width of concrete foundation
8 0	1.1	2.54	2.25	16.20	12–3	1.5	3.554	2.25	23.54
8 6	1.1	2.67	2.25	17.08	13–6	1.8	3.82	2.25	25.64
9 6	1.5	2.92	2.25	18.84	14–9	1.8	4.07	2.25	27.70
9 9	1.5	2.98	2.25	19.26	15–0	1.8	4.11	2.75	28.36
10 3	1.5	3.10	2.25	20.14	15–9	1.8	4.26	2.75	29.58
11 6	1.5	3.38	2.25	22.26	16–6	1.8	4.39	2.75	30.78

arch determined by drawing arc of circle through these two points, this arc having its center directly below the center of the sewer. Below springing line, wall had batter of one horizontal to four vertical.

Other sections of various diameters were constructed along general plan of section shown; principal dimensions of several are given in Table 129. The invert in each case was lined with two rings of brick, a total of about 9 in. in thickness.

These sections are noteworthy for heavy masonry to retain line of resistance within middle third of section at all points and to spread thrust on soil to reduce soil pressure to not more than 2 tons per square foot. Sections also noteworthy on account of construction of arch and the unusual bonding of the brickwork adopted as productive of a much more stable structure than would result from use of ordinary bond. *Trans. Am. Soc. C. E.*, 1905; **55**, 401.

Figure 123*a*.—Worcester, Mass., Sewer Dept., 38- by 50-in. brick, egg-shaped sewer, typical of construction used extensively in many old systems throughout the country. In recent years, however, this type has been replaced largely by sections shown in Figs. 123*c*, *d*, *e*, and *f*. Many of these old sewers show but few signs of distortion due to earth pressures. Where this type was built on steep grades in combined systems, the invert bricks have been worn to a considerable extent and, in some cases, worn through, causing backfilling and supporting earth outside brickwork to be washed away and resulting in caving in of sewer. This trouble overcome by making invert masonry heavier and lining invert with hard-burned or vitrified brick, calculated to resist wear better.

Figure 123*b*.—Worcester, Mass., Sewer Dept., 48- by 72-in. brick, egg-shaped sewer, interesting on account of special shape used in several instances in that city.

Figure 123*c*.—Borough of Brooklyn, New York City, 1901, H. R. Asserson, Chief Engineer. 54-in. brick, egg-shaped sewer, with two types of construction. This sewer was designated by the size of the equivalent circular sewer instead of by dimensions of the egg-shaped section.

Figure 123*d*.—Borough of Brooklyn, New York City, Bureau of Sewers, 1913, E. J. Fort, Chief Engineer. Standard 36-in. egg-shaped sewer of much interest when compared with Fig. 123*e*.

Figure 123*e*.—Philadelphia, Pa., 1906, standard sections, George S. Webster, Chief Engineer. Egg-shaped sewer, 2 ft. 8 in. by 4 ft. Left half, construction in firm material when minimum section can be used; right half, construction called "reduced" cradle. Piles 12-in. yellow pine 3 ft. apart longitudinally and 3 ft. 4 in. transversely.

Figure 123*f*.—Philadelphia, Pa., standard sections, 1906, George S. Webster, Chief Engineer. 2-ft. 8-in. by 4-ft. egg-shaped sewer. Left half, construction in "maximum" cradle on piles, piles 12 in. in diameter spaced 3 ft. longitudinally and 2 ft. 6 in. transversely. Right half, construction on timber platform and piles; platform 6-in. yellow-pine planking on 8- by 8-in. yellow-pine stringers on 12-in. yellow-pine piles 3 ft. apart longitudinally and 2 ft. 7 in. apart transversely. Where sewers are on steep grades, inside below springing line has one ring of vitrified shale brick.

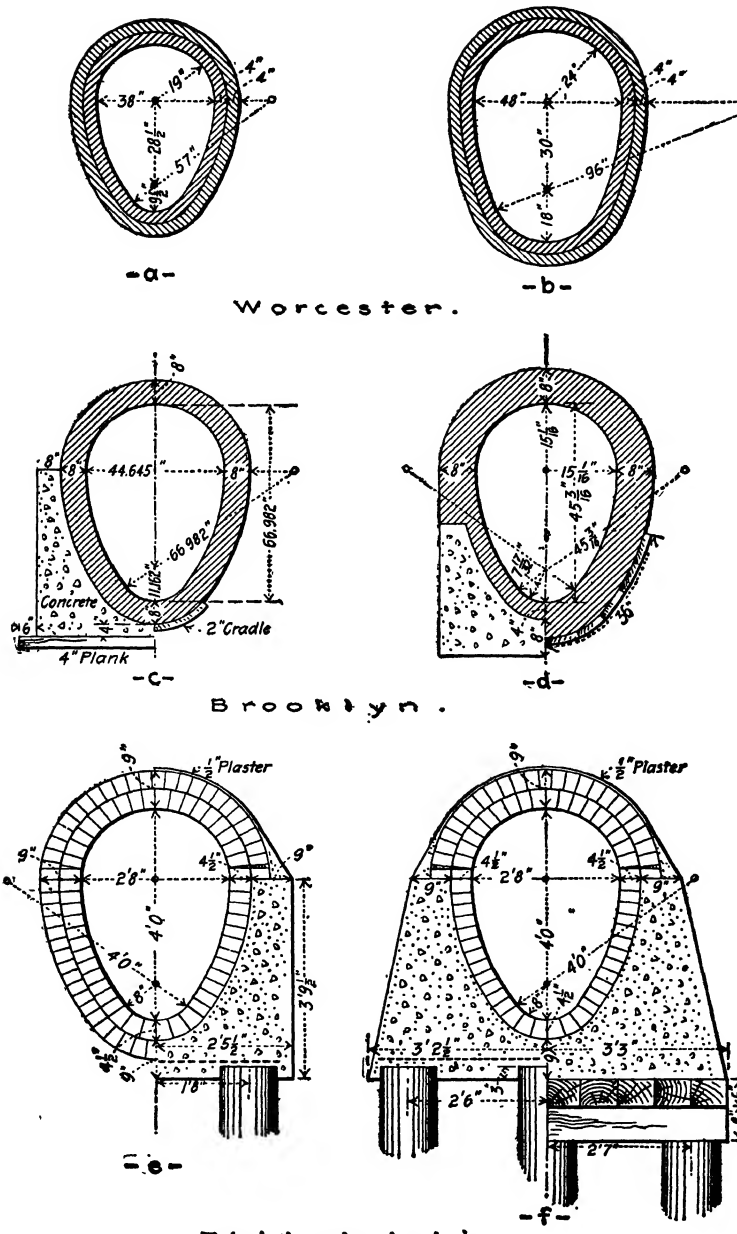


FIG. 123.—Typical egg-shaped sections.

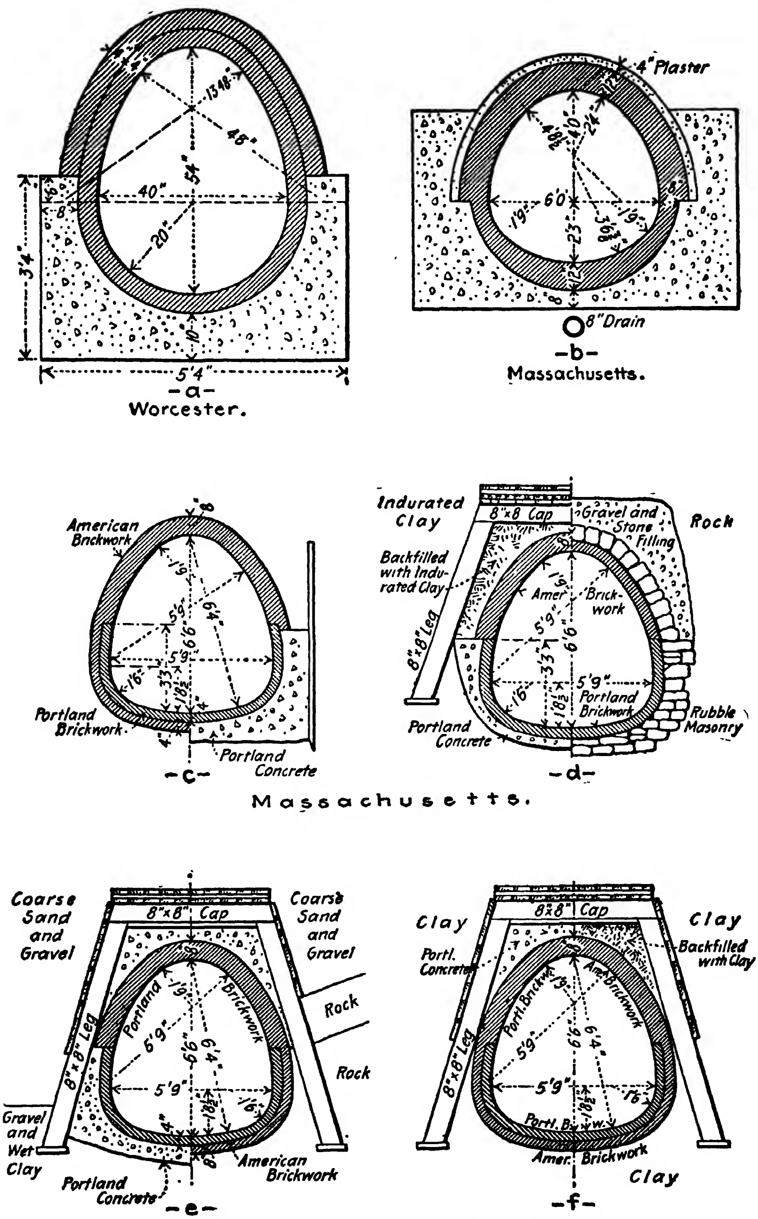


FIG. 124.—Typical inverted egg-shaped and catenary sections.

Figure 124*a*.—Worcester, Mass., Sewer Dept., 1899, H. P. Eddy, Supt. Water St. 40- by 54-in. inverted egg-shaped intercepter. Average depth to crown of sewer, 17 ft. Sewer constructed in tunnel, largely rock but partly earth roof requiring bracing. Section chosen for its economy of space with wooden timbering and the additional inside head room available.

Figure 124*b*.—Massachusetts Metropolitan Sewerage Com'n, 1891, Howard A. Carson, Chief Engineer. Sec. 1, North Metropolitan Outfall Sewer, Deer Island near pumping station. Catenary 6- by 6¼-ft. type. Average depth of cover about 8 ft. Section designed to act under slight head; brick arch made extra heavy to produce excess of downward pressure. *Eng. News*, 1894; **31**, 121.

Figures 124*c*, *d*, *e*, and *f*.—Massachusetts Metropolitan Sewerage Com'n, North Metropolitan Sewer, Sec. 26, 1892, Howard A. Carson, Chief Engineer. Catenary 5¾- by 6½-ft. section. Conditions generally permitted building invert in excavation without special foundation. Nearly half distance was in clay permitting an all-brick section, but balance was in clay, sand, or gravel, requiring various forms shown. The entire length was protected by a timber platform with clay or concrete backfill between platform and sewer, except about 300 ft. where section 124*e* right half was used. Average depth of fill for sections in open cut, about 17 ft. Average depth above crown of sewer to surface of ground for tunnel section, about 24 ft. *Eng. News*, 1894; **31**, 121.

Figure 125*a*.—Washington, D. C., main conduit near pumping station, 1905, designed in office of Engineer Commissioner, District of Columbia. Oval 9- by 7-ft. 2-in. sewer. Short length of section connects main 6- by 6-ft. horseshoe sewer with trunk sewer and discharges into cunette in section shown in Fig. 135*c*.

Figure 125*b*.—Washington, D. C., low area trunk sewer, 1905, designed in office of Engineer Commissioner, District of Columbia. Oval 4- by 6-ft. sewer. About 100 ft. built. This section and Fig. 125*a* selected to fulfill special requirements.

Figure 125*c*.—Chicago, Ill., Western Ave. sewer, 1910, Isham Randolph, Chief Engineer, Sanitary District of Chicago. Elliptical 12- by 14-ft. sewer. Excavation generally in stiff blue clay, average cover 10 ft. Reinforcement used only in section 400 ft. long, under railroad yards, cover 4 to 5 ft. No reinforcement in city streets. Under Illinois and Michigan Canal, section changed to 12- by 9-ft. ellipse for distance of 60 ft. long with 5-ft. fall in that length. *Eng. Contr.*, 1910; **33**, 405; 1914; **51**, 20.

Figure 125*d*.—Massachusetts Metropolitan Sewerage Com'n, North Metropolitan Sewer, Sec. 41, 1892, Howard A. Carson, Chief Engineer. Elliptical sewer, 1 ft. 8 in. by 2 ft. 6 in. Average cover about 10 ft. Excavation in sand, gravel, ledge, boulders, filling, and very fine sand containing much water. In places, the fine sand was removed to 1 ft. below bottom of sewer and replaced with gravel. In other places, piles averaging 25 ft. were driven, bents 2 ft. on centers, with 8- by 10-in. caps and 2-in. flooring. Ledge was replaced by tamped gravel for 6 in. below bottom of brickwork. In sand, excavation carried to firm foundation and the brickwork bedded in and surrounded by gravel. In fine running sand, sewer laid in cradle of 1-in. boards on 2- by 4-in. ribs, and cradle covered with broken stone. Cradle lined with tar paper. Another section had cradle of two thicknesses of boards with tar paper between. *Eng. News*, 1894; **31**, 121.

Figure 125*e*.—Altoona, Pa., 1896. Oval sewer, 33¼ by 44 in. Section had one-ring brickwork and 4 to 8 in. concrete, with invert of vitrified-shale paving brick. Cost claimed to be less than cost of two-ring brick sewer. *Proc. Eng. Club Philadelphia*, 1897; 91.

Figure 125*f*.—Richmond, Va., 1912. False elliptical 8- by 10-ft. and 8- by 12-ft. sections, chosen on account of insufficient depth for circular sewer. Curves of arch and invert were three-centered, with row of headers at point of change of radius to tie the rings together. On account of shallow cover, buttresses were built every 12 ft. to give arch good bearing against sides of ditch. Double-track railroad crosses sewer with only about 4-ft. cover. Portion constructed in 4- to 8-ft. rock cut, where concrete invert lined with one ring of brick and arch of three rings of brick were used. Figure 125*f* shows one-half of each of the two sizes. *Eng. & Contr.*, 1912; **38**, 579.

Figure 126*a*.—Massachusetts Metropolitan Sewerage Com'n, North Metropolitan Sewer, Sec. 14, 1892, Howard A. Carson, Chief Engineer. Basket-handle section, 8 ft. 2 in. by 8 ft. 10 in. Left half of section, construction in firm material where bottom could be shaped to invert; right half, construction on timber platform on piles. Platform was 4-in. plank floor on 10- by 12-in. caps, on piles spaced 2 ft. 7 in. centers transversely. *Eng. News*, 1894, **33**, 121.

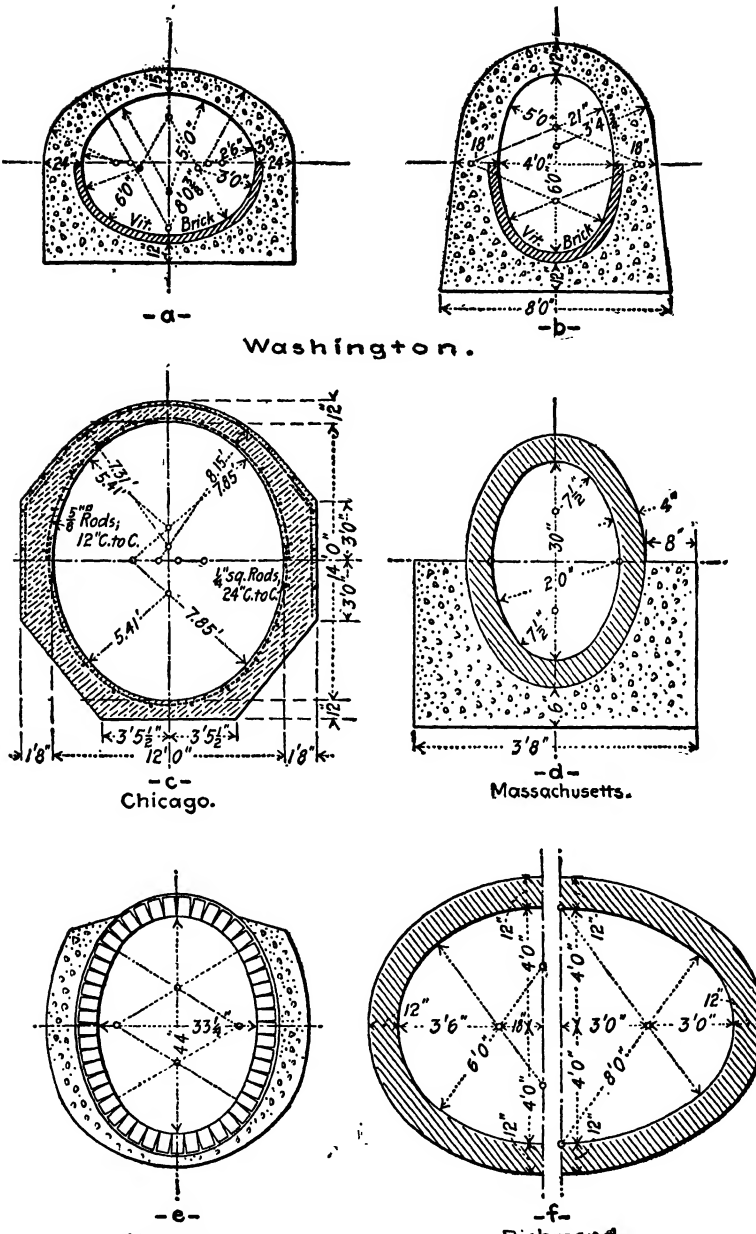


FIG. 125.—Typical elliptical sections.

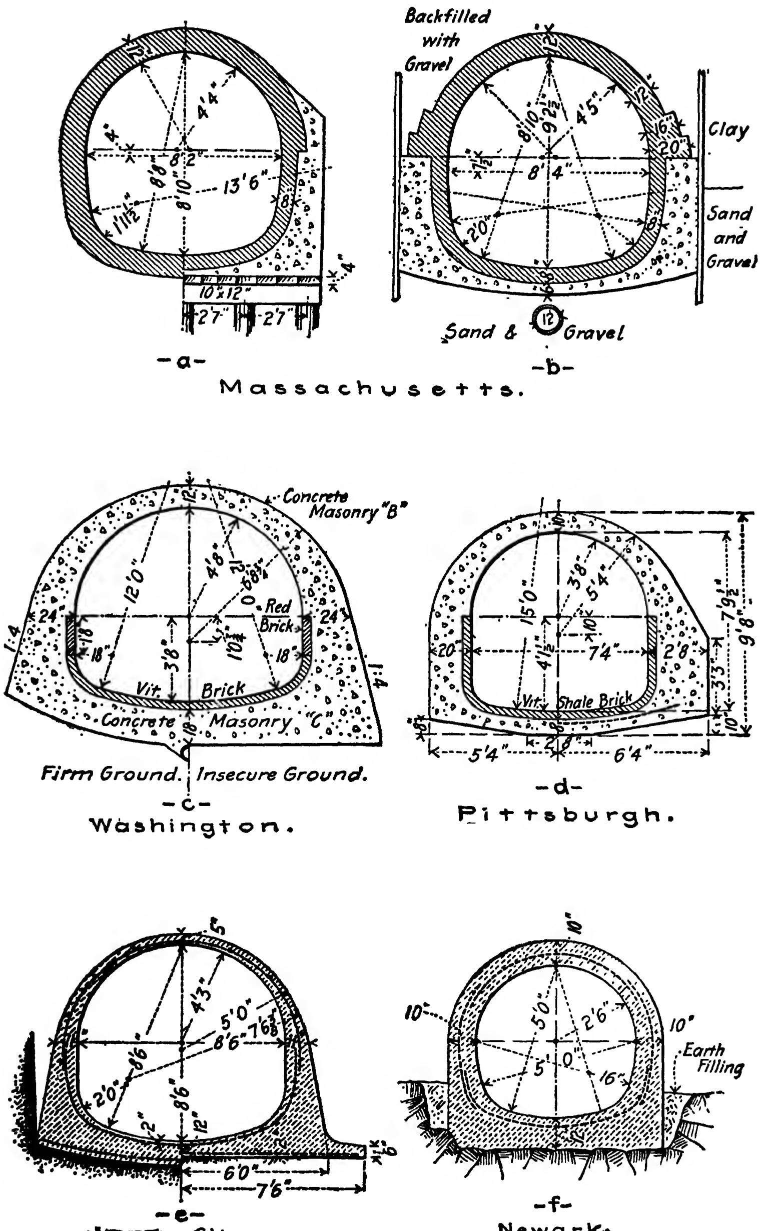


FIG. 126.—Typical basket-handle sections.

Figure 126*b*.—Massachusetts Metropolitan Sewerage Com'n, North Metropolitan Sewer, Contract Sec. 14, 1892. Howard A. Carson, Chief Engineer. Basket-handle sewer, 8 ft. 4 in. by 9 ft. 2½ in., used where material below springing line was sand and gravel and that above was clay. Sewer arch backfilled with gravel. *Eng. News*, 1894; **31**, 121.

Figure 126*c*.—Washington, D. C., Outfall Sewer, 1904, designed in office of Engineer Commissioner of District of Columbia. Basket-handle section, 9 ft. 4 in. by 8 ft. 4 in. Left half, construction in firm ground; right half, construction in yielding soil or in insecure ground. Several hundred feet on piles, masonry section same as right half of figure. Pile spacing, one in center, one on either side 3 ft. 7⅛ in. from center, and one outside pile on each side 3 ft. 4 in. from center of next adjacent pile, making five piles to bent, bents spaced 3 ft. 6 in. c. to c. Another section built on 3-in. yellow-pine floor on 10- by 12-in. yellow-pine caps on bents containing six piles, spaced 2 ft. 8 in. on centers.

Figure 126*d*.—Pittsburgh, Pa., Try St. Drainage Sewer, Bureau of Surveys, Charles M. Reppert, Division Engineer. Basket-handle section 7 ft. 4 in. by 7 ft. 9½ in. Left half, construction for firm ground; right half, construction for soft foundation. A 6-ft. 8-in. by 7-ft. ½-in. section was also constructed, 9 in. thick at crown and 18 in. at springing line for firm-ground section and 30 in. for soft-ground section; and invert below vitrified shale brick lining, 8 in. thick. Maximum width of latter sewer, 9 ft. 8 in. and 11 ft. 8 in. for firm- and soft-ground sections, respectively. A 5-ft. 8-in. by 5-ft. 11½-in. section was 8 in. thick at crown and 16 in. at springing line for firm-ground section and 28 in. for soft-ground section. Thickness of invert below vitrified shale brick lining, 6 in.; maximum width of masonry, 8 ft. 4 in. and 10 ft. 4 in., respectively, for the firm- and soft-ground sections.

Figure 126*e*.—Jersey City Water Supply Co., Jersey City, N. J., aqueduct, 1903, E. W. Harrison, Chief Engineer. Basket-handle section, 8 ft. 6 in. by 8 ft. 6 in. Left half of illustration, construction in soft earth; right half, section built on embankment. Where cover was about 15 ft., arch was 8 in. thick at crown and side walls 14 in. thick at springing line. *Eng. Record*, 1904; **49**, 72.

Figure 126*f*.—Newark, N. J., Water Dept., inlet conduit in reservoir, 1901; Morris R. Sherrerd, Engineer. Basket-handle section, 5 by 5 ft. Outlet from reservoir comprises two conduits similar to one shown placed side by side, wall between two 10 in. thick and space between extrados of sections filled with concrete. Maximum width of double-conduit section, 12 ft. 6 in. Both single and double conduit sections have comparatively heavy walls to provide sufficient dead weight to overcome buoyancy of conduits when empty and reservoir full. Test section of double conduit subjected to hydrostatic pressure up to 34 lb. per square inch without signs of weakness. *Eng. Record*, 1903; **48**, 725.

Figure 127*a*.—Wachusett Aqueduct, Mass. Metropolitan Water Works, 1897, F. P. Stearns, Chief Engineer. Horseshoe section, 11 ft. 6 in. by 10 ft. 6 in. The figure shows construction in rock cut, and by full and dotted lines the types in earth from hardpan to soft foundations. Cover shallow; about 4 ft. for a considerable distance. *Eng. News*, 1897; **37**, 114.

Figure 127*b*.—Hartford, Conn., aqueduct, 1912. C. M. Saville, Chief Engineer. Horseshoe section, 4 ft. 9 in. by 5 ft. Largely in earth trench with about 3-ft. cover.

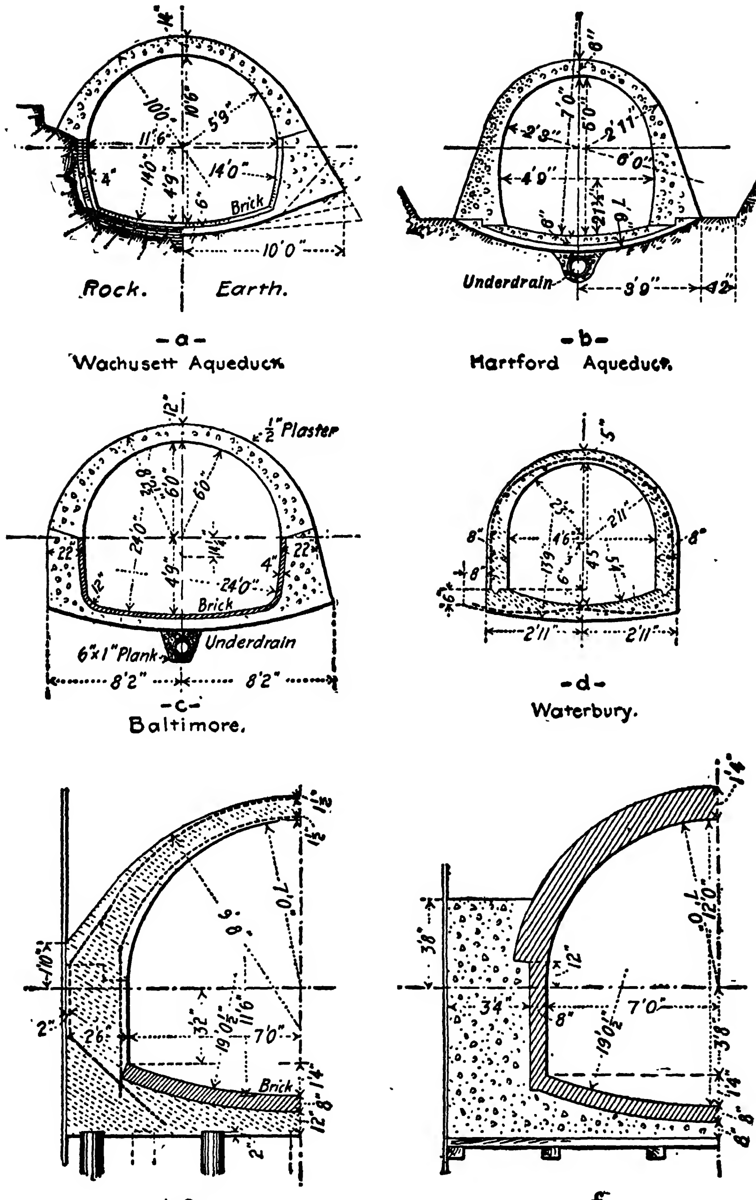
Figure 127*c*.—Baltimore, Md., Outfall Sewer, 1907. Calvin W. Hendrick, Chief Engineer. Horseshoe shape, 12 ft. by 10 ft. 9 in. Left half, construction used in tunnel or sheeted trench; right half, type in loose earth or fill. *Eng. Record*, 1908; **57**, 163.

Figure 127*d*.—Waterbury, Conn., main intercepting sewer, 1907, R. A. Cairns, City Engineer Horseshoe shape, 4 ft. 6 in. by 4 ft. 5 in. On soft bottom footing extended 8 in. outside vertical walls. About 1,500 ft. in river bed constructed with much heavier section forming retaining wall. *Eng. Record*, 1908; **57**, 466.

Figure 127*e*.—Boston, Mass., Tenean Creek Conduit, 1909, E. S. Dorr, Chief Engineer. Horseshoe shape, 14 ft. by 11 ft. 6 in. The conduit was constructed on piles, 4 to a bent placed 5 ft. c. to c.

Figure 127*f*.—Boston, Mass., Tenean Creek Sewer. Brick horseshoe conduit, 14 by 12 ft. This is much older than Fig. 127*e* and affords an interesting comparison between the former methods, involving the use of a brick arch with concrete backing, and the modern type of reinforced concrete construction. Structure built on timber platform of 4-in. plank on 6- by 8-in. sills.

Figure 128*a*.—Cambridge, Mass., Marginal Conduit, 1908, Charles River Basin Com'n, Hiram A. Miller, Chief Engineer. Horseshoe section, 6 by 5 ft.



Boston

FIG. 127.—Typical horseshoe sections.

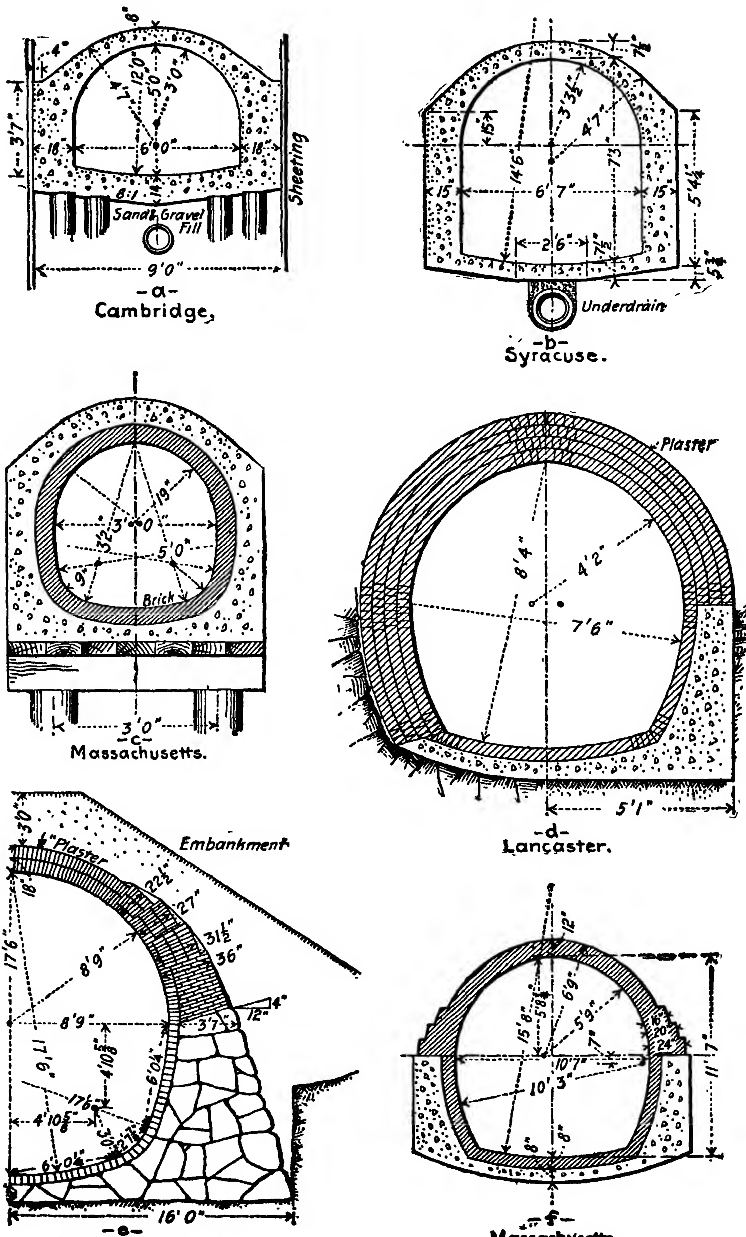


FIG. 128.—Typical horseshoe sections.

Figure 128*b*.—Syracuse, N. Y., Main Intercepting Sewer, 1910, Intercepting Sewer Board, Glenn D. Holmes, Chief Engineer. Horseshoe section, 6 ft. 7 in. by 7 ft. 3 in., equivalent to 87-in. circular. Smaller sections built of same general form with thinner masonry. The 4-ft. 10-in. by 5-ft. 4-in. section had 6-in. crown and invert thickness and 10-in. side-wall thickness.

Figure 128*c*.—Massachusetts Metropolitan Sewerage Com'n, North Metropolitan Sewer, Sec. 22, 1892, Howard A. Carson, Chief Engineer. Horseshoe or basket-handle section, 3 ft. by 3 ft. 2 in. Constructed generally in very fine running sand on 3-in. plank platform on 8- by 8-in. caps, on piles 3 ft. c. to c., two piles to bent. For short distance on clay foundation, sewer built on cradle of 1-in. boards laid on 2- by 4-in. ribs; constructed entirely of two rings of brick masonry. *Eng. News*, 1894; **31**, 121.

Figure 128*d*.—Lancaster, Pa., 1903, Samuel M. Gray, Engineer. Horseshoe section, 7 ft. 6 in. by 8 ft. 4 in. The type shown in left half contains 32.6 cu. ft. brickwork and 4.8 cu. ft. concrete per linear foot; type shown in right half contains 24.7 cu. ft. brickwork and 18.5 cu. ft. concrete. Sectional area of waterway, 50.4 sq. ft. If constructed of concrete, sections could be reduced to 7 ft. 4 in. by 8 ft. 2 in. with the same general shape. Concrete section in rock, thickness was 6 in. at crown, 9 in. at springing line, and 6 in. at invert below vitrified brick lining. Section reinforced with 3-in. No. 10 expanded metal. Section contained 2 cu. ft. of brickwork, and area of waterway was 49.06 sq. ft.

Figure 128*e*.—Philadelphia, Pa., Annsbury St. Sewer, 1909, George S. Webster, Chief Engineer, Bureau of Surveys. Horseshoe section, 17 ft. 6 in. by 17 ft. 6 in. Built generally in shallow cut with 3-ft. cover over the top of the sewer. Fig. 122*b* shows another type of brick construction of interest in comparison with that in this figure.

Figure 128*f*.—Massachusetts Metropolitan Sewerage Com'n, South Metropolitan High Level Sewer, 1902, William M. Brown, Chief Engineer. Horseshoe type, 10 ft. 7 in. by 11 ft. 7 in. Concrete used generally for side walls and invert backing, with one or two rings of brick lining, depending upon amount of groundwater. Concrete occasionally used for arch, but arches were mostly 12-in. brickwork.

Figure 129*a*.—Louisville, Ky., Beargrass Intercepter, Sec. A., 1908, J. B. F. Breed, Chief Engineer. Horseshoe section, 6 ft. 6 in. by 6 ft. 1½ in. Left half, construction in open cut with 3- to 11-ft. cover; the right half, type in tunnel. Excavation in clay and sand; water encountered in open cut. One section built on piles driven about 20 ft. in bents of three each, 4 ft. on centers. Portion of tunnel section built on concrete piles, in holes bored with augur, making the finished hole 10 in. in diameter. Material encountered, a fill of clay and mud. Vertical steel reinforcement placed in each hole and hole then filled with concrete. Some material encountered was so wet and mucky, that concrete was placed through iron casing withdrawn as concrete filled hole. In another section, 12-in. wrought-iron pipe casing was driven, and concrete placed in it without reinforcement. Most tunnel work was in dry, loose, running sand. The entire cross-section of the tunnel was backfilled with concrete to a point 1 ft. above springing line of sewer arch.

Figure 129*b*.—Louisville, Ky., 34th St. Outlet Sewer, 1909, J. B. F. Breed, Chief Engineer. Horseshoe section, 7 ft. by 6 ft. 8 in. Maximum cover, about 25 ft.; average, about 10 ft. Excavation largely in sand, gravel, and clayey loam with some loose rocks. Interior of sewer below springing line lined with vitrified brick. Structure built for considerable distance on Simplex concrete piles.

Figure 129*c*.—Louisville, Ky., Northwestern Sewer, Sec. B1, Contract No. 53, 1910, J. B. F. Breed, Chief Engineer. Horseshoe shape, 13 ft. 6 in. by 9 ft., equivalent to 11-ft. 3-in. circular sewer. Excavation mainly in sand and gravel with some yellow clay.

Figure 129*d*.—Louisville, Ky., Northwestern Sewer, Sec. B2, Contract No. 54, 1910, J. B. F. Breed, Chief Engineer. Horseshoe section, 9 by 9 ft. Excavation in clay and sand. Average cover about 13 ft.

Figure 129*e*.—Borough of the Bronx, New York City. Horseshoe shaped, 8 ft. 6 in. by 6 ft. 9½ in.; very heavy construction for soft foundation.

Figure 129*f*.—New Bedford, Mass., Outfall Sewer, 1912, William F. Williams, City Engineer. Horseshoe section, 7 ft. 8 in. by 7 ft. Right half, construction for 2- to 8-ft. cover; left half, heavier section for more severe loading. Materials per linear foot of sewer, 27.3 cu. ft. concrete, 84 lb. reinforcing bars, for cover from 2 to 5 ft., and 88.65 lb. for cover from 5 to 8 ft., for left half. Materials per linear foot of sewer, 34.75 cu. ft. concrete, 90.6 lb. of reinforcing steel for right half.

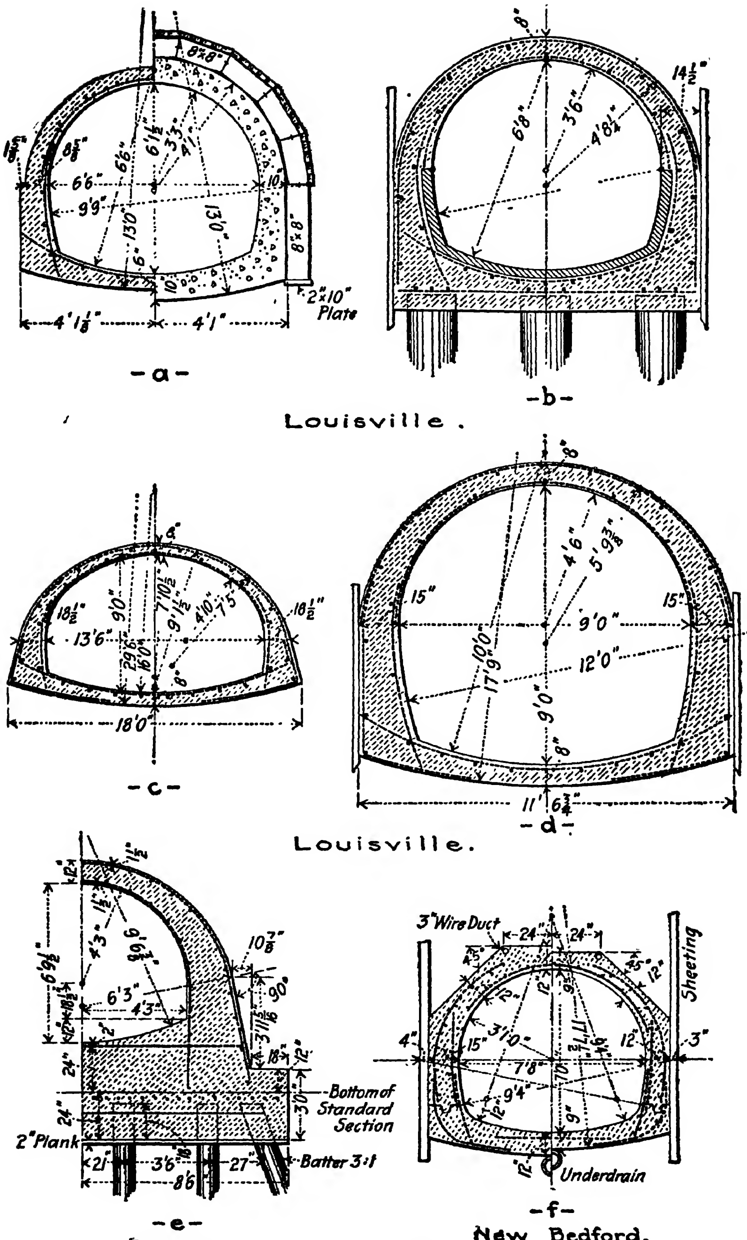


FIG. 129.—Typical horseshoe sections.

Figure 130*a*.—Philadelphia, Pa., Torresdale Filtered-water Conduit. Semielliptical section, 9 by 9 ft. to stand 20-ft. head of water. REID, "Concrete and Reinforced-concrete Construction," p. 663.

Figure 130*b*.—Syracuse, N. Y., Main Intercepting Sewer, 1910, Glenn D. Holmes, Chief Engineer. Semielliptical section, 7 ft. 7¼ in. by 7 ft. 5 in.

Figure 130*c*.—Catskill Aqueduct, Board of Water Supply, New York City, 1908, J. Waldo Smith, Chief Engineer. Semielliptical type, 17 ft. 6 in. by 17 ft. Cut shows construction in earth cut; dotted lines on invert show extension of section when structure was built on embankment.

"The aqueduct in dry loose earth was designed to withstand the weight of the embankment about it, whether full or empty, and also to withstand the water pressure when full without the aid of the surrounding embankment; it was designed to withstand the pressure due to the water's rising, from some unusual condition, above the inside top of the arch. With the regular 3-ft. embankment over the top of the arch, the cut-and-cover sections are safe to carry a 12-ton road roller, a condition that may occasionally occur at road crossings. The section is strong enough to withstand a fill not over 14 ft. deep over the top of the arch. For fills greater than this, reinforcement of steel rods will be placed in the invert to enable it to withstand the reaction caused by the heavy load. In cases where a wet earth foundation is encountered, the aqueduct will be constructed on a timber platform arranged to allow the groundwater to drain away to sumps without washing away the freshly laid concrete. Wherever the level of the groundwater adjacent to the aqueduct is higher than 9 ft. above the invert, the latter is to be made thicker, in order to withstand the upward hydrostatic pressure when the aqueduct is empty. The section in compact earth was designed to effect a lower cost per linear foot of aqueduct where the character of the earth warrants, by making the bottom width narrower, by steepening the slopes of the excavation, and by laying the concrete directly against the earth sides. This section can, of course, be used only where the earth is compact enough to take the thrust of the concrete arch without yielding. The section on embankment is similar to that in loose earth, except that in order to lessen the danger of settlement the base is made wide enough to distribute the load over a larger area. Provision is also made for a foundation embankment more carefully constructed than the rest of the embankment and for a possible reinforcement of the invert in such cases. The section in rock was designed so that the rock will nowhere extend nearer than 12 in. to the inside surface of the aqueduct, thus insuring stability and water tightness. Provision was made in the designs for using excavated rock in parts of the embankment at the sides and top of the aqueduct." *Rept.* of Board, 1907.

Figure 130*d*.—Philadelphia, Pa., Mill Creek Sewer, 1912, George S. Webster, Chief Engineer. Semielliptical or parabolic type, 18 ft. 5 in. by 17 ft. 6 in.

Figure 130*e*.—Boston, Mass., Charles River Basin Com'n, Marginal Conduit, 1905, Hiram A. Miller, Chief Engineer. Semielliptical section, 6 ft. 4 in. by 7 ft. 8¼ in. Part constructed on gravel and clay bottom and remainder on piles 2 ft. apart on centers under the side walls and 4 ft. apart under the center of the invert.

Figure 130*f*.—Louisville, Ky., Southern Outfall Sewer, Sec. E, Contract No. 14, 1909, J. B. F. Breed, Chief Engineer. Semielliptical section, 12 ft. 3 in. Average cover about 21 ft. Material excavated, sand and gravel overlaid with considerable alluvial clay; no groundwater.

Figure 131*a*.—Borough of the Bronx, New York City. Semicircular concrete sewer, 11 ft. 6 in. by 7 ft. 3 in. Piles driven in bents of five vertical and two brace piles, one on either side; bents spaced 3 ft. 6 in. to 4 ft.; piles in bent spaced 3 ft. 3 in. c. to c. *Trans. Am. Soc. C. E.*, 1913; **76**, 1784.

Figure 131*b*.—Wilmington, Delaware, Clement's Run Sewer, 1903. T. Chalkley Hatton, Consulting Engineer. Semicircular sewer 10 ft. by 5 ft. 6 in., reinforced with woven wire mesh and No. 6 expanded metal. Invert lined with one course of brick. *Munic. Eng.* 1904; **27**, 248.

Figure 131*c*.—Borough of Brooklyn, New York City, 1913, E. J. Fort, Chief Engineer. Semicircular section, 8 ft. 4 in. by 5 ft. 8 in., equivalent to 78-in. circular section. Table 130 gives a comparison of the hydraulic properties of the semicircular section and of a 78-in. circular section, both at the maximum capacity of the section.

Figure 131*d*.—Boston, Mass., Kemp St. Overflow, 1912, E. S. Dorr, Chief Engineer. Semicircular section, 10 ft. 3⅝ in. by 6 ft. 3 in.

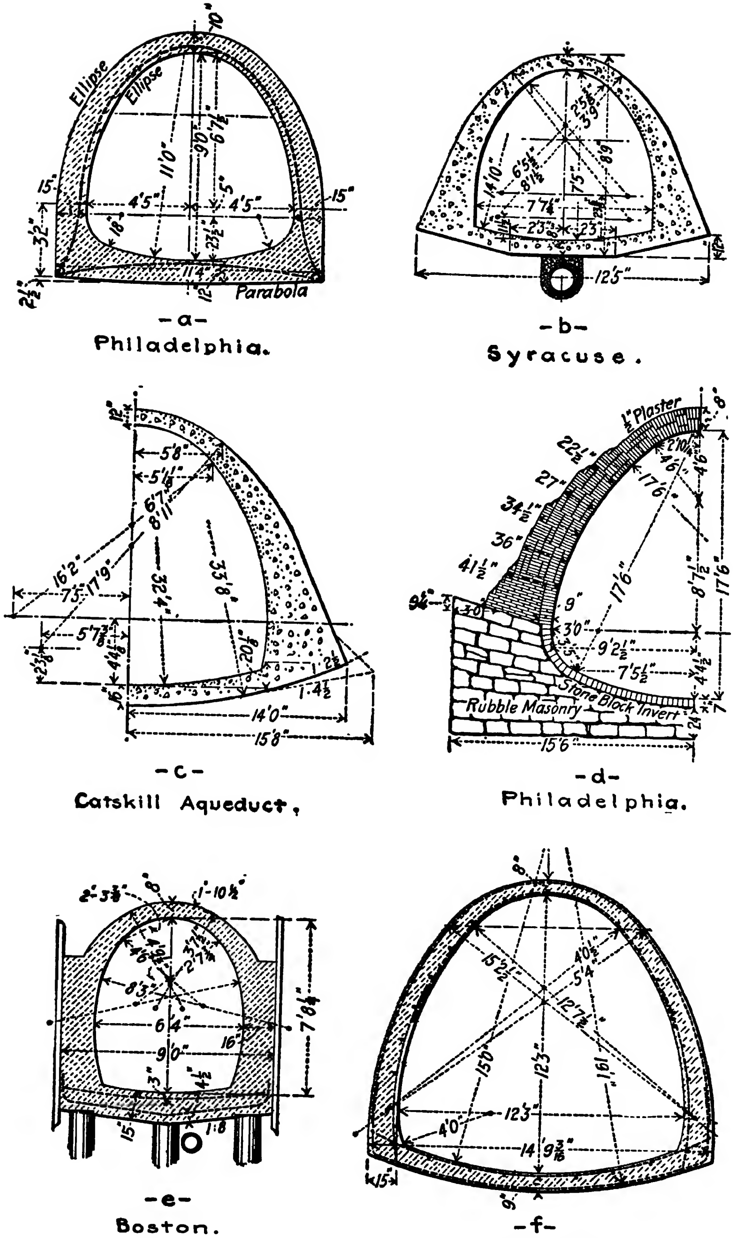


Fig. 130.—Typical semi elliptical sections.

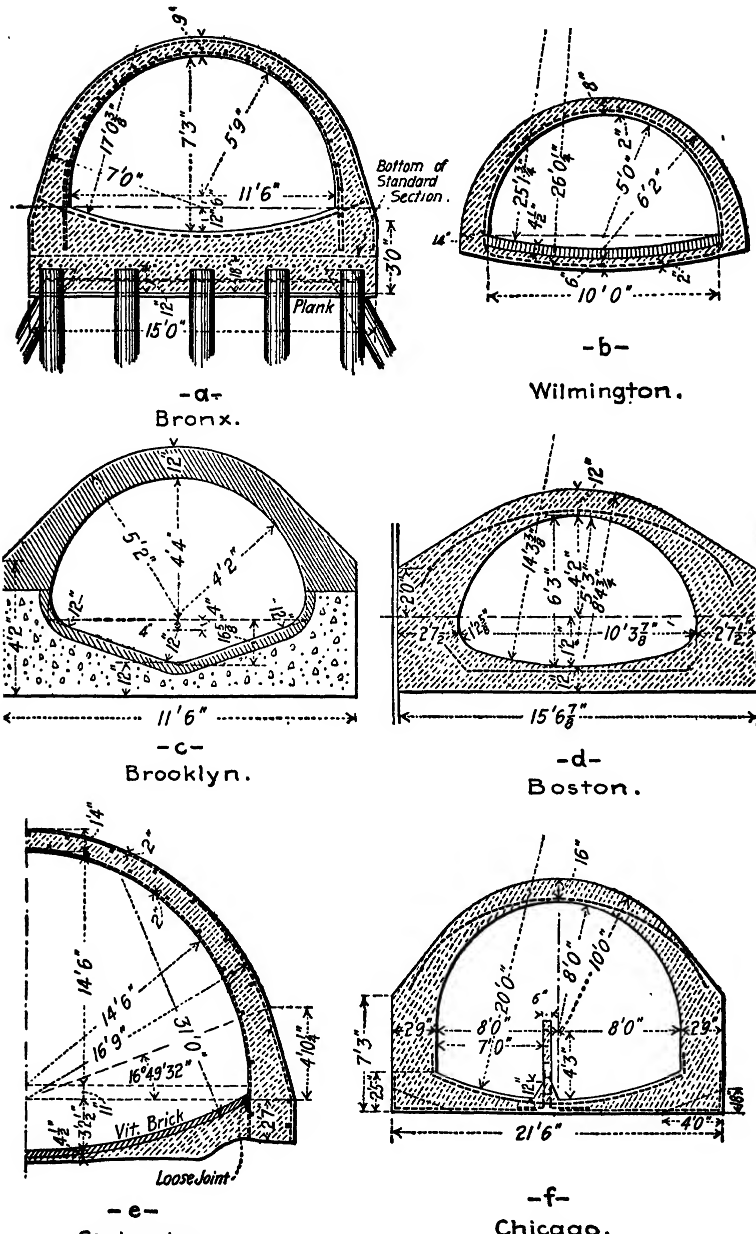


FIG. 131.—Typical semicircular sections.

TABLE 130.—COMPARISON OF SEMICIRCULAR AND CIRCULAR SECTIONS

Section	Area, square feet	Wetted perimeter, feet	Hydraulic radius, feet	Discharge $S = 0.001$ c.f.s.
Semicircular..........	32 94	18.20	1.81	151.31
Circular..................	32.35	17.20	1.93	156.25

Figure 131*e*.—St. Louis, Mo., Harlem Creek Sewer, 1908, H. F. Fardwell, Sewer Commissioner. Semicircular section, 29 ft. by 18 ft. 7½ in., to carry 15-ft. fill over arch and the heaviest railroad loading combined with 7-ft. fill. Stresses in various sections determined from analysis of circular ribs with fixed ends given in Prof. Charles E. Greene's "Trusses and Arches." Arch carried through to rock, and loose joint left between side wall and invert. In earth, section considerably widened at base, invert much thicker and reinforced to distribute thrust of arch over greater area. *Eng. Record*, 1907; **56**, 664.

Figure 131*f*.—Chicago, Ill., Sanitary District, South 52d Ave. Sewer, 1913, George M. Wisner, Chief Engineer. Horseshoe section, 16 ft. by 12 ft. 3 in. Dividing wall is to provide high velocities and avoid deposits by keeping dry-weather flow on one side of the wall until such time as the flow is large enough to use the whole section. Stop planks at head of section divert flow to either side of dividing wall. Owing to soft ground, invert reinforced throughout entire length. Joint between dividing wall and invert strengthened by two sets of bars bent at right angles. Height of wall above invert, 4 ft. 11 in.; wall slightly off center. *Eng. & Contr.*, 1914; **41**, 201.

Figure 132*a*.—St. Louis, Mo., South Harlem Joint District Sewer, 1909. Horseshoe section, 12 ft. by 9 ft. 7¼ in. Left half, section in rock cut; right half, section for earth. Arch designed for 20-ft. cover. In earth, materials per linear foot were concrete, 2.62 cu. yd.; vitrified brick, 0.174 cu. yd ; transverse reinforcement, 90.2 lin. ft. ⅝-in. bars; longitudinal reinforcement, 22 lin. ft. ½-in. bars. For section in rock, materials were concrete, 1.92 cu. yd.; vitrified brick, 0.174 cu. yd.; transverse steel, 73.7 lin. ft. ⅝-in. bars; longitudinal steel, 19.0 lin. ft. ½-in. bars.

Figure 132*b*.—St. Louis, Mo., Dale Ave. Sewer, 1910. Rectangular section, 6 ft. 3 in. by 8 ft. Types were designed to meet three conditions. In first, natural rock surface was at or above skewback of flat arch, which had to carry whole load directly to rock. The 9-in. concrete walls were merely to smooth up sides of cut. In second case, rock was slightly below skewback, and 18-in. concrete walls used. In third case, rock was more than 3 ft. below springing line; see right half of figure. The 18-in. walls were designed as beams to carry arch thrust at upper end and earth pressure below. As sewer was largely in rock, the narrow, high rectangular section was selected as most economical. Owing to depth at which sewer was built, saving in excavation due to decreased width much more than offset increased depth. *Eng. News*, 1912; **68**, 426.

Figure 132*c*.—St. Louis, Mo., Baden Public Sewer, First Section, 1910. Five-centered, or semielliptical arch section, 18 ft. by 13 ft. 1⅜ in. Left half, construction in rock cut; right half, type for earth cut. W. W. Horner states that use of this type, instead of that shown in Fig. 132*a*, has been a matter of judgment in each particular case. The five-centered arch has been preferred where loading was principally uniform earth load.

Figure 132*d*.—St. Louis, Mo. Horseshoe section, 16 ft. 6 in. by 16 ft. 6 in. Left half, construction where rock was encountered above springing line; right half, construction in earth cut. Arch designed to carry 25-ft. fill above crown.

Figure 133*a*.—Harrisburg, Pa., Paxton Creek Intercepting Sewer, 1903, James H. Fuertes, Consulting Engineer. Parabolic section, 6 by 5 ft.; also, smaller section of same type, 5 ft. 1½ in. by 3 ft. 9 in., with same thickness of masonry. Sewer crosses swamp and meadow land mainly. Probably first parabolic section in this country. Loaded coal train was derailed on siding directly over sewer within 2 weeks after completion without injuring it. Backfill very wet clay; top of sewer about 5 ft. below track. At other points no ill effects resulted from pressure of 20 ft. very wet backfill. *Eng. Record*, 1904; **50**, 444.

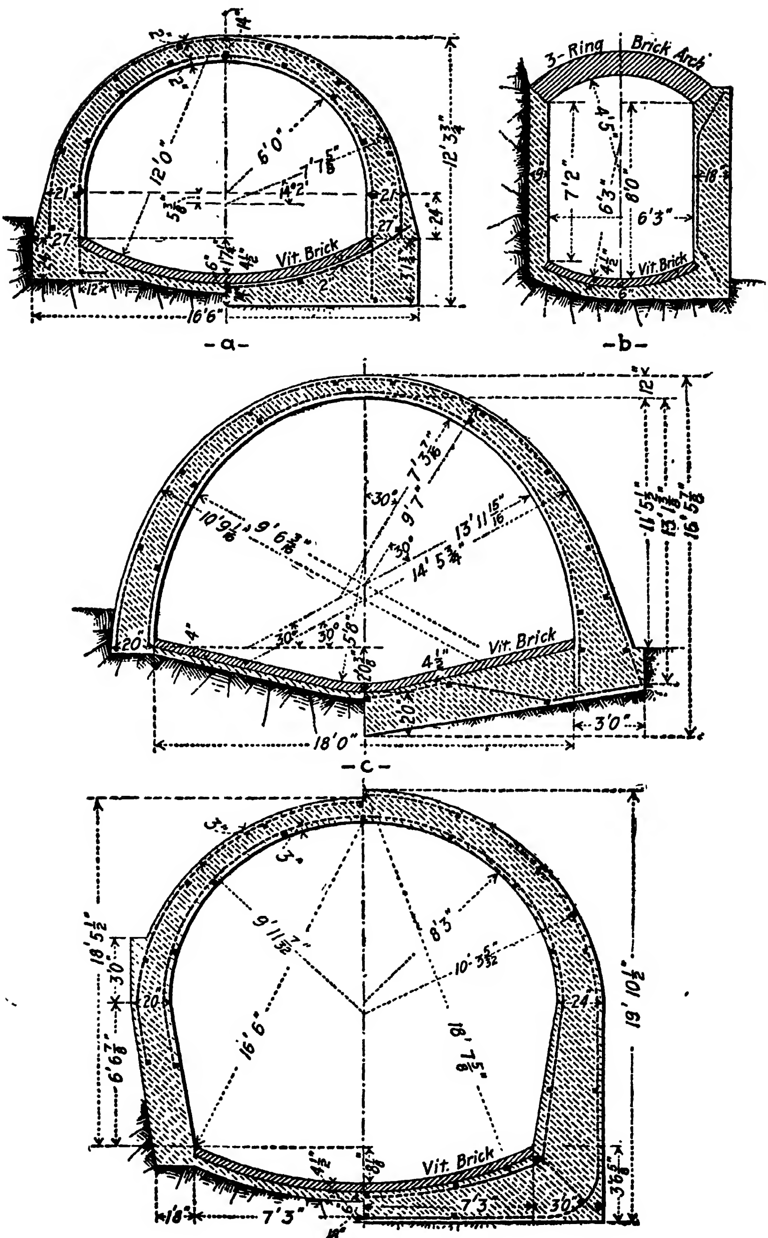


FIG. 132.—Typical St. Louis sections.

Figure 133*b*.—Louisville, Ky., Happy Hollow Sewer, Contract No. 1, 1907, J. B. F. Breed, Chief Engineer. Parabolic section, 8 ft. 6 in. by 7 ft. 4 in., built in shallow cut, in places more than half the sewer being above natural surface of ground. Excavation in loam and clay. Section considered especially advantageous for conditions, on account of economy of space in embankment section and strong arch afforded. Sewer may be covered by fill of 30 ft. hereafter.

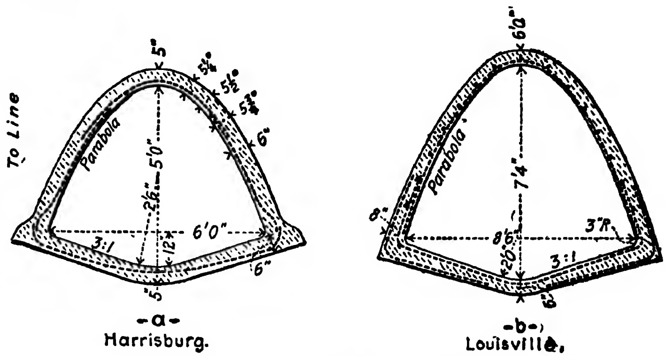


FIG. 133.—Typical parabolic or delta sections.

Figure 134*a*.—Beargrass Creek Drain, Sec. A, Contract No. 36, 1909, J. B. F. Breed, Chief Engineer. Rectangular section, 6 ft. by 4 ft. 9 in., constructed in alluvial clay requiring foundation of oak piles driven in bents of three, spaced 3 ft. 2 in. c. to c.

Figure 134*b*.—Harrisburg, Pa., Susquehanna River Intercepting Sewer, 1912, James H. Fuertes, Consulting Engineer. Rectangular section, 3 ft. 6 in. by 3 ft. 6 in. Rectangular section chosen on account of proximity of sewer to surface of ground. *Eng. Record*, 1912; **65**, 218.

Figure 134*c*.—Ogden, Utah, 1907, A. F. Parker, City Engineer. Rectangular conduit, 3 ft. 9 in. by 2 ft. 7⅞ in., with top practically at surface of ground in street and located so that gutter and curb form part of conduit. Under street crossings, side walls reduced in height to 16⅛ in. and the top curved in form of arch like invert. *Eng. Record*, 1908; **57**, 65.

Figure 134*d*.—Des Moines Ia., Ingersoll Run Sewer, 1905, John W. Budd, City Engineer. Rectangular section, 10 by 5 ft., is selected on account of proximity of grade line of sewer to street surface. *Eng. Record*, 1906; **53**, 537.

Figure 134*e*.—Borough of Brooklyn, New York City, 1913, E. J. Fort, Chief Engineer. Rectangular section 10 ft. by 6 ft. 8 in., approximately equivalent to 102-in. circular sewer. Table 131 compares the hydraulic properties of the rectangular section, filled to within 12 in. of the crown, with the properties of a 102-in. circular sewer.

TABLE 131.—COMPARISON OF HYDRAULIC PROPERTIES OF RECTANGULAR AND CIRCULAR SECTIONS

Section	Wetted area, sq. ft.	Wetted perimeter, feet	Hydraulic radius, feet	Discharge in cu. ft., sec., $S = 0.001$
10 ft. × 6 ft. 8 in. rectangular	49.12	18.55	2.65	294.60
102 in. circular	55.43	22.51	2.41	319.64

Figure 134*f*.—Hoboken, N. J., 1913, James H. Fuertes, Consulting Engineer. Rectangular section, 7 ft. by 4 ft. 9 in., is of particular interest on account of V-shaped waterway pro-

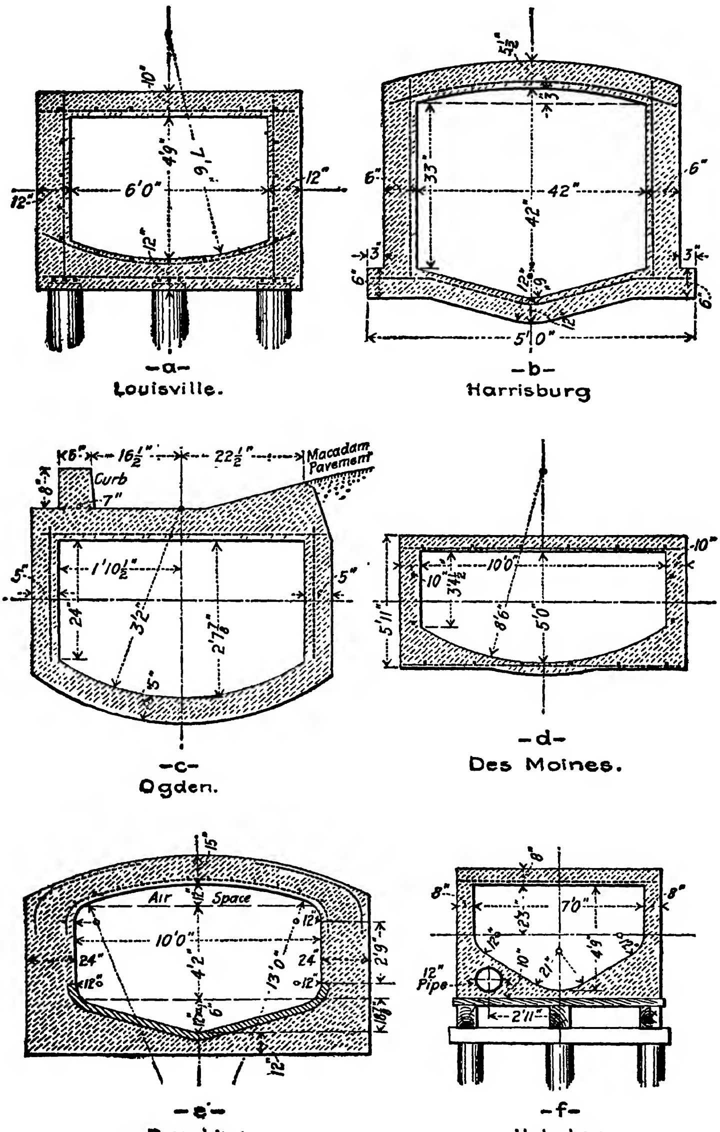


Fig. 134.—Typical rectangular sections.

vided for low flows. Rectangular section with flat top selected on account of lack of head room between surface of ground and top of sewer. Sewer in soft foundation has timber platform of 4-in. planks on 10- by 12-in. stringers on 3- by 8-in. caps, two to each pile bent. Piles spaced 3 ft. 8 in. c. to c., three piles to a bent.

Figure 135*a*.—Lancaster, Pa., 1903, Samuel M. Gray, Engineer. Semicircular, 12- by 6-ft., section with 24-in. half-round dry-weather flow channel or "cunette." Two types designed, one with concrete arch reinforced with 3-in. mesh No. 10 expanded metal and the other with an arch of four rings of brickwork.

Figure 135*b*.—Louisville, Ky., Southern Outfall Sewer, Sec. A, Contract No. 6, 1908, J. B. F. Breed, Chief Engineer. Horseshoe section, 8 ft. wide with 3-ft. half-round cunette. Section on incline of about 30 deg. to horizontal, and cunette used to carry dry-weather flow on account of high velocity. Invert of cunette lined with 36-in. vitrified-clay channel pipe.

Figure 135*c*.—Washington, D. C., New Jersey Ave. Trunk Sewer, 1902, designed in office of Engineer Commissioner of District of Columbia. Semicircular, 18- by 16-ft. section with 9-ft. half-round cunette.

Figure 135*d*.—Brussels, Belgium, Maelbeek Creek Storm-water Sewer, 1895. Horseshoe shape, 14 ft. 9½ in. by 12 ft. 1¾-in., with 6-ft. 8-in.-wide cunette. Interior of sewer lined with ¼-in. cement plaster, and exterior covered with ¾-in. coating of cement mortar. *Eng. News*, 1896; **35**, 195.

Figure 135*e*.—Syracuse, N. Y., Harbor Brook Intercepting Sewer, 1912, Glenn D. Holmes, Chief Engineer. U-shaped 30-in. section. Left half, construction in firm material; right half, section on pile foundation. Flat slab top built separately and set in place, joints being filled with mortar.

A sewer of practically same design 3 ft. wide at top was constructed in Lynn, Mass., in 1909, from plans of C. H. Dodd, Chief Draftsman, Boston Sewer Department, who also designed similar section, 2 ft. wide on top, for Boston, in 1908.

Figure 135*f*.—Borough of Richmond, New York City, District 6A Trunk Sewer, 1907, Louis L. Tribus, Commissioner of Public Works. U-shaped 6-ft. 6-in. semicircular section. General surface of land below top of sewer. *Eng. Record*, 1907; **56**, 486.

Figure 136*a*.—Salt Lake City, Big Cottonwood Water Conduit, 1907, L. C. Kelsey, City Engineer. Rectangular section, 3 ft. 5 in. by 4 ft. 5½ in. Figure shows construction in fill; similar section used in excavation, except reinforcing bars were placed nearer interior. In tunnel, section resembled that shown but lacked reinforcement. *Eng. Contr.*, 1908; **30**, 78.

Figure 136*b*.—Philadelphia, Pa., Devereaux St. Sewer, 1909, George S. Webster, Chief Engineer. Section constructed in mud through lowland on 2½- by 5-ft. piers spaced 15 ft. c. to c. longitudinally and 11 ft. 6 in. apart transversely. Sewer protected by embankment with 3-ft cover. Between invert and pier were three vertical dowels 1 in. square, 12 in. c. to c. Roof pitched 2 in. from center to outside, plastered with 1-in. cement mortar.

Figure 136*c*.—Boston, Mass., South End Sewer Improvement, Sec. 2, Union Park St., 1913, E. S. Dorr, Chief Engineer. Double-conduit rectangular sections, 6 ft. 5 in. by 6 ft. 5 in., and 6 ft 5 in. by 4 ft. 2 in. Double structure required by limited space for construction of conduits. Section constructed on platform of 2-in. plank laid on 3- by 4-in. sills.

Figure 136*d*.—Boston, Mass., South End Sewer Improvement, Sec. 4. Albany St.; 1913; E. S. Dorr, Chief Engineer. Gravity sewer, 4 ft. 10 in. by 10 ft 6 in ; force main 2 ft. 9 in. by 10 ft. 6 in.; rectangular sections, double conduit. Section constructed on 4-in. planks on 8- by 8-in. caps on three-pile bents. Section shows limits to which it is sometimes necessary to go where space is very much restricted.

Figure 136*e*.—Boston, Mass., Stony Brook Channel, 1908, E. S. Dorr, Chief Engineer. Double section, 8 ft. 3 in. by 10 ft. 6 in., constructed to replace old stone masonry channel, and on that account work involved special difficulties. Section with I-beams in roof used, that backfilling might be placed more quickly than on section reinforced with bars. This type laid on platform of 1-in. boards on 2- by 3-in. sills.

Figure 137*a*.—Borough of the Bronx, New York City, Broadway Outfall Sewer, 1902, J. A. Briggs, Chief Engineer, C. H. Graham, Engineer of Sewers. Twin semicircular section, 15 ft. by 9 ft. 2⅛ in., constructed largely aboveground, twin section being adopted as requiring less vertical space than single large circular sewer. Depth of cover to surface of street, 4 ft. Sewer constructed on concrete, timber, rubble, or pile foundations, depending upon character of soil. *Eng. Record*, 1905; **52**, 550.

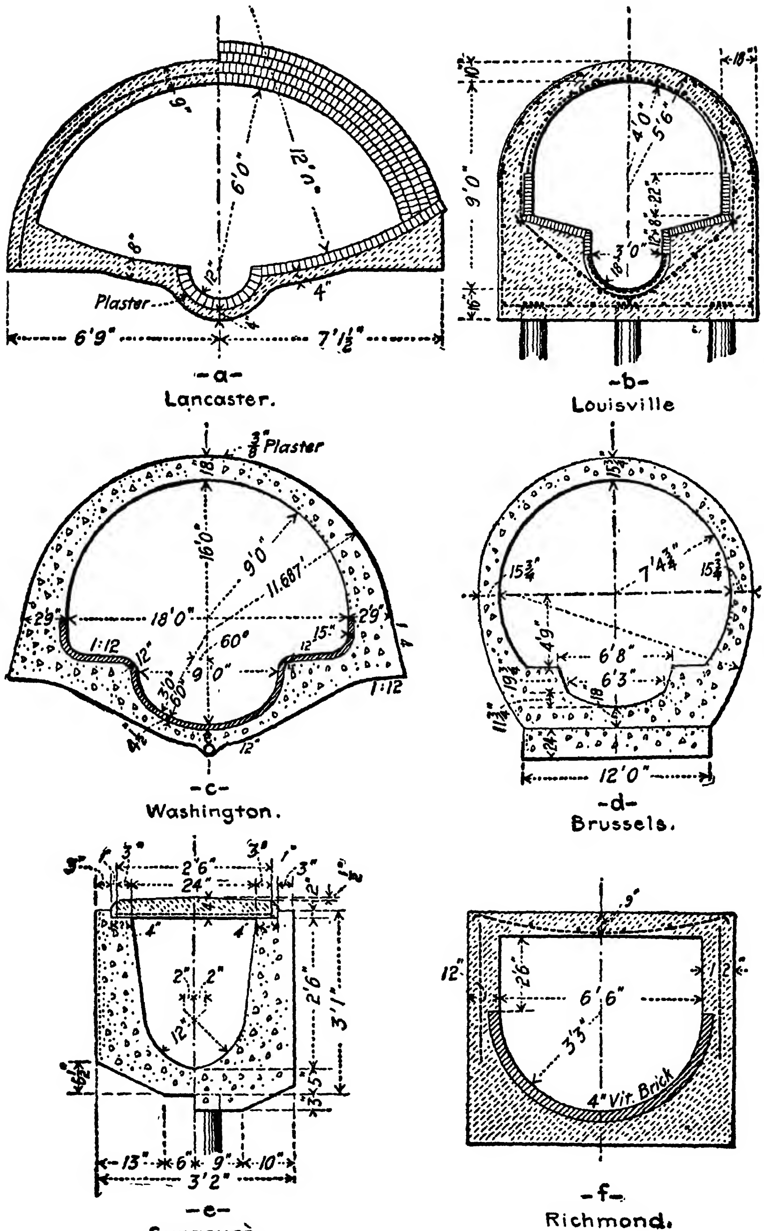


Fig. 135.—Cunette and U-shaped sewer sections.

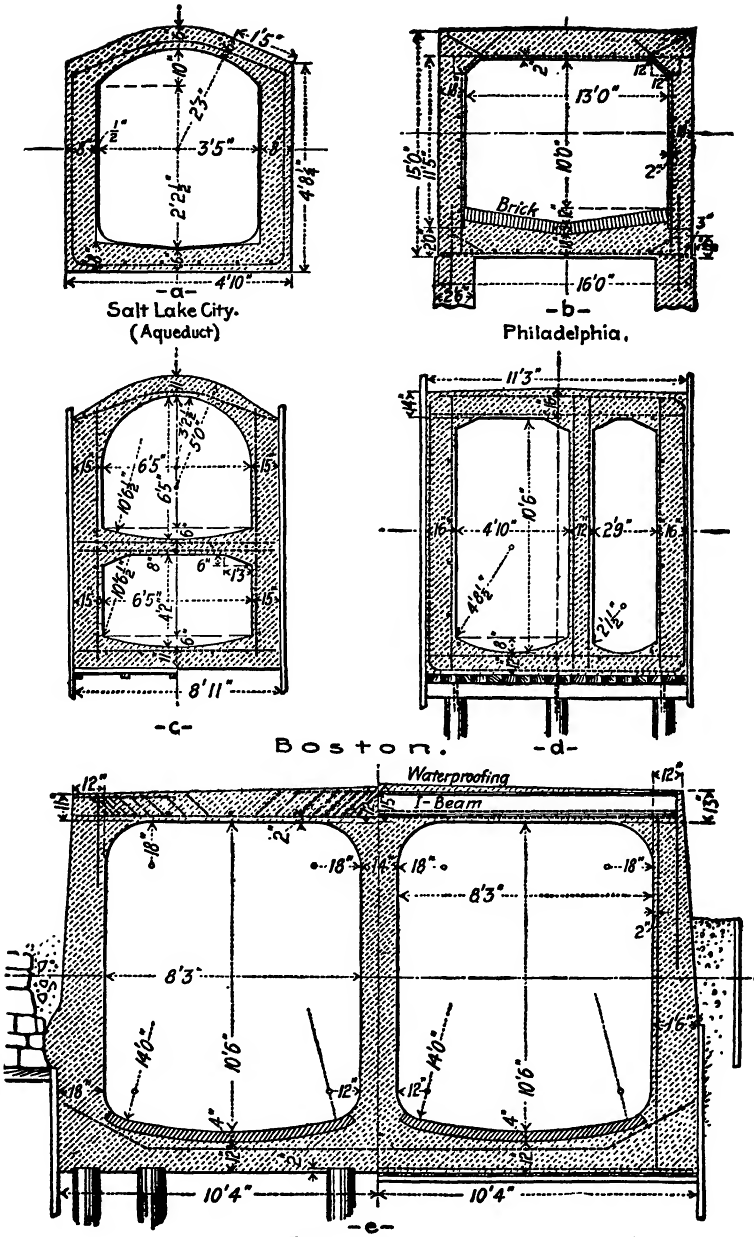
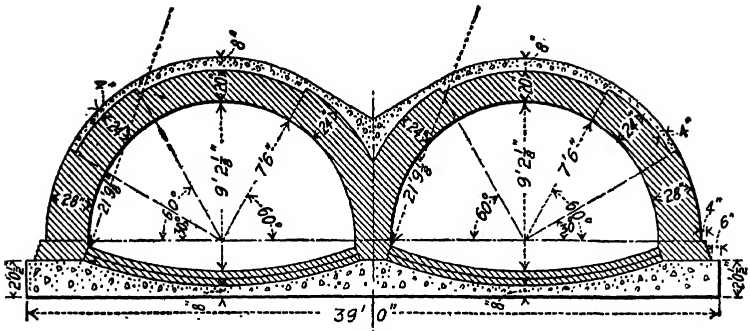
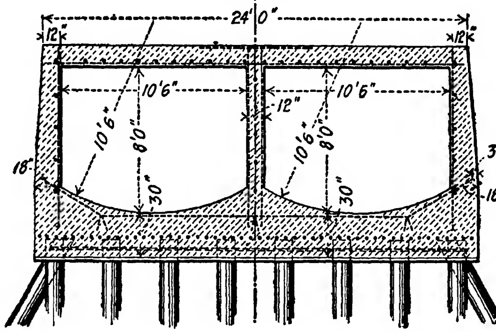


FIG. 136.—Typical rectangular sections.





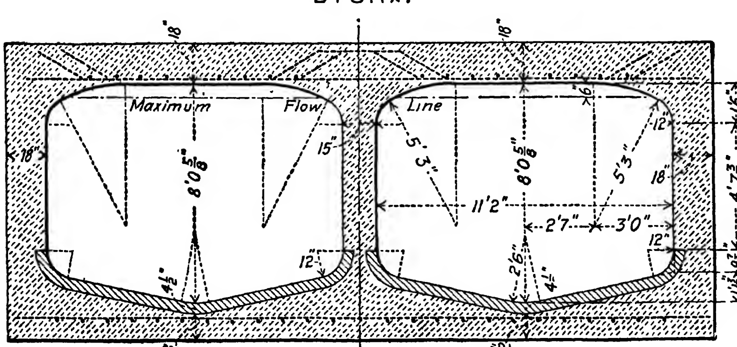
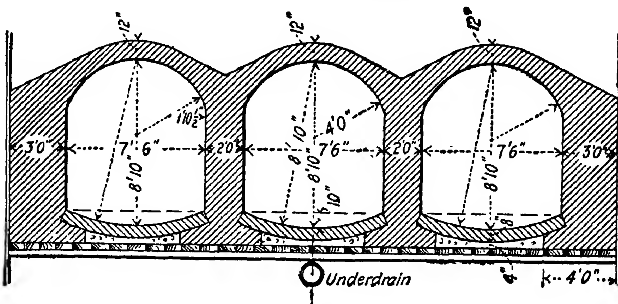


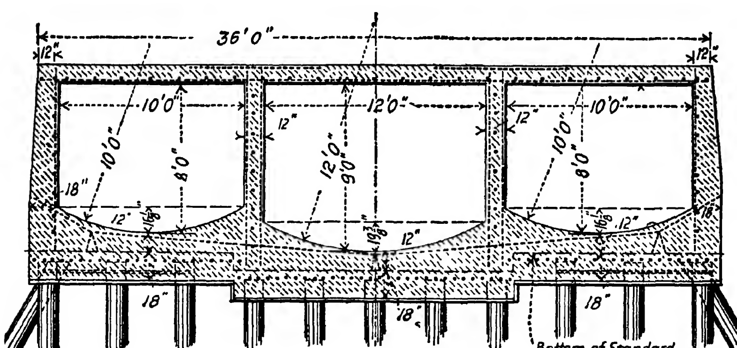
FIG. 137.—Typical double sections.

Figure 137*b*.—Borough of the Bronx, New York City. Rectangular twin section 10 ft. 6 in. by 8 ft. 10 in. Piles spaced 3 ft. 3 in. c. to c.; 8 vertical piles to bent with two brace piles, one on either side. *Trans. Am. Soc. C. E.*, 1913; **76**, 1784.

Figure 137*c*.—Borough of Brooklyn, New York City, 1913, E. J. Fort, Chief Engineer. Twin rectangular section, 11 ft. 2 in. by 8 ft. 5⅝ in., approximately equivalent to 13-ft. circular sewer. Flowing completely full, rectangular section estimated to discharge 908.80 cu. ft. per second and 13-ft. circular sewer 908.01 cu. ft. on a slope of 0.001. Twin rectangular section discharging at maximum flow line, allowing 6-in. air space at top of each channel, discharges approximately 1037.53 cu. ft. per second, as against 980.65 cu. ft. per second—the maximum of the 13-ft. circular sewer. There is material saving in head room with the twin rectangular section over the equivalent circular section.



-a-
Brooklyn.



-b-
Bronx.

FIG. 138.—Typical triple sections.

Figure 138*a*.—Borough of Brooklyn, New York City, 64th St. Outfall Sewer, 1901, H. R. Asserson, Chief Engineer. Triple rectangular section, 7 ft. 6 in. by 8 ft. 10 in.

Figure 138*b*.—Borough of the Bronx, New York City. Triple sewer, one 12- by 9-ft. and two 10- by 8-ft., rectangular sections. Sewer constructed on bents with 11 vertical and 2 brace piles each, bents spaced 4 ft. c. to c. The concrete around piles deposited on 2-in. plank platform.

Figure 139.—Louisville, Ky., Northeastern Sanitary Trunk Sewer and Beargrass Creek Drain, Contract No. 36, Sec. A, 1909, J. B. F. Breed, Chief Engineer. Compound structure including 5-ft. 7½-in. by 4-ft. semicircular sanitary sewer and 6-ft. by 4-ft. 9-in. rectangular

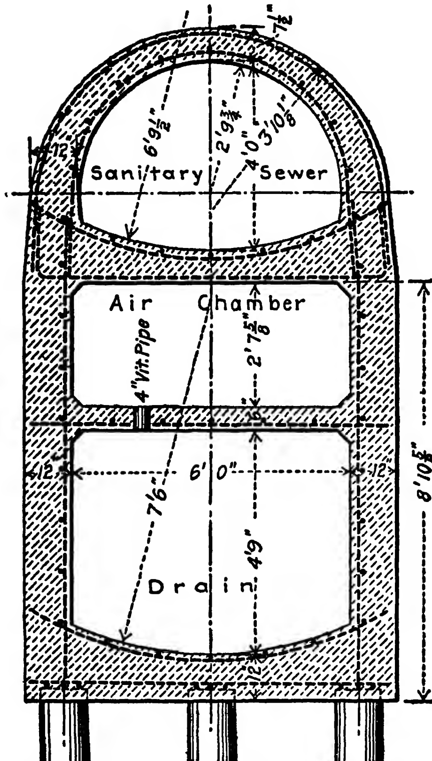


Fig. 139—Compound sewer section, Louisville.

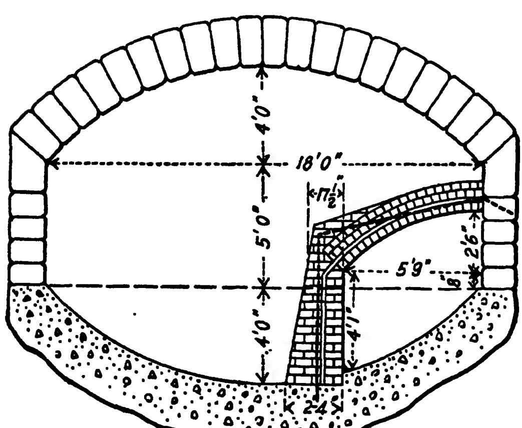


Fig. 140.—Millbrook intercepter, Worcester.

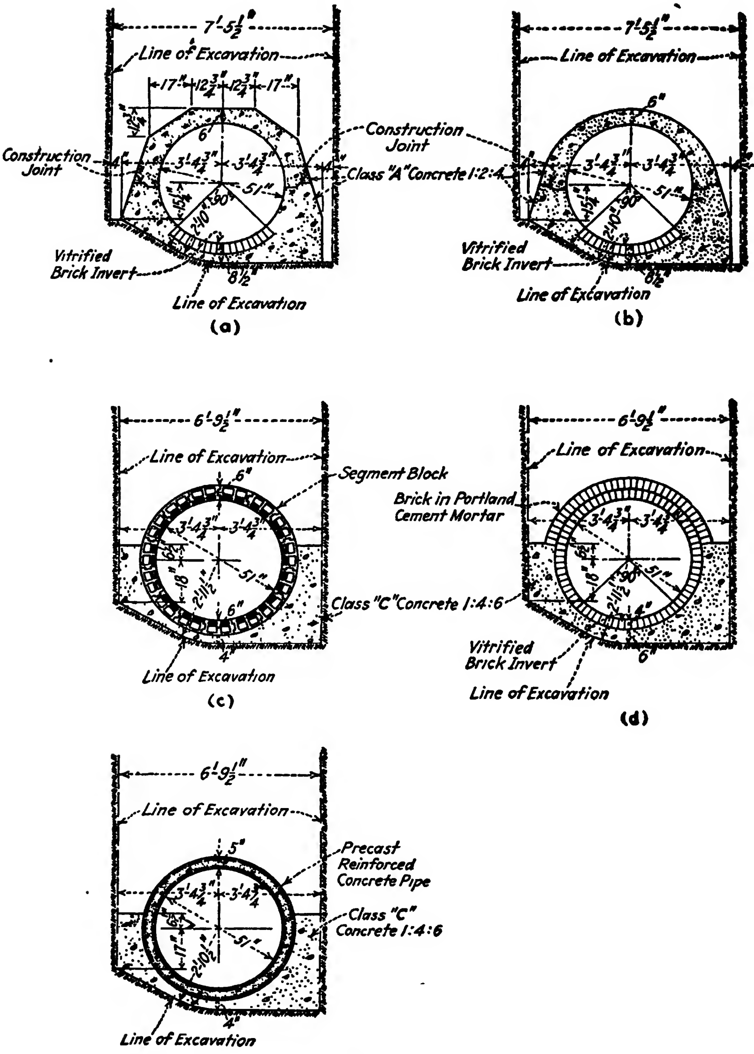


Fig. 141.—Alternate sections of 51-inch sewer.

drain, separated by an air space or chamber. Relative positions of channels due to low bearing power of soil, requiring a pile foundation, and the resulting economy in using one set of piles for both structures. Piles spaced 3 ft. 2 in. c. to c. transversely, three piles to bent. Fall of drain and sewer toward opposite ends; sewer and drain were separated at upper end by very low chamber, which gradually increased, due to increasing difference in elevations of sewer and drain. Material excavated almost wholly alluvial clay. Bents 3 ft. c. to c. Structure built in four operations, invert of drain first, after which the side walls and top of the drain were constructed. Following the completion of the drain, the side walls of the air chamber between the sewer and the drain and the invert of the sewer were built as a third operation, after the completion of which the concrete was placed in the arch of the sewer.

Figure 140.—Worcester, Mass., Millbrook Intercepting Sewer, 1897, Frederick A. McClure, City Engineer. Larger section is old trunk sewer constructed in 1880 of quarried stone with concrete invert, laid through ledge and occupying so much of street that it was deemed impracticable to parallel it with intercepters. Accordingly, conduit was designed to accommodate flow of sewage inside large sewer. The brick section was constructed inside cofferdam; average depth of flow in main sewer during construction, about 3 ft. See annual *Rept.* of Superintendent of Sewers, Worcester, 1899.

Provision for Forms.—There are cases where it may be of advantage to design the sewer section to accommodate the forms to be used in its construction. Experienced sewer contractors have usually been willing to build concrete sewer arches with a curved exterior surface, thereby saving in masonry, at a lower total cost per foot than with plane surfaces. Occasionally, however, it has been found economical to form the exterior surface by planes, as shown in Figs. 129*f* and 141*a*.

If support for the exterior arch forms is desired, it can be had by providing a narrow ledge at or near the springing line, as shown in Fig. 129*a* (left half). Similar provision can be made on the inside, but the break in the smooth interior surface is objectionable from a hydraulic standpoint.

SELECTION OF MATERIALS OF CONSTRUCTION

Materials for Arches.—In the older sewerage systems will be found examples of large sewer arches constructed of *stone blocks*. An example is shown in Fig. 140, a section of the Millbrook conduit in Worcester, Mass. One reason for choosing stone blocks was their availability and lower cost as compared with brickwork for large arches; in the days when such sewers were constructed, concrete and reinforced concrete were used little, if at all. Even more recently, rubble masonry has been used to a considerable extent, especially in Philadelphia, on account of its relative economy. Its use, however, has been largely for foundations and masonry below the springing line. Stone blocks have now practically been superseded by other materials for sewer arches. Although stone arches have fewer joints, it is more difficult to obtain tight work, and consequently the leakage is apt to be larger than when brick are used.

Brick masonry is still used to some extent for sewer arches, principally on account of its economy in certain cases and the ease with which brick masonry can be handled in tunnels and restricted places. Brick arches, owing to their greater number of joints, are more liable to deformation than concrete, and unless special means are employed in bonding the brick the full strength of the arch may not be developed. Yet owing to the great number of joints, a brick arch may sometimes readjust itself by deformation without developing serious cracks. Moreover, contraction and other cracks, if any exist, are likely to be small and at frequent intervals. Since there is no reinforcing metal, cracks do not afford opportunity for corrosion of such metal with resulting reduction of strength of the structure.

In the construction of brick arches, three general types of bonding have been used. In the first, the arch is built of concentric rings of brick with all bricks laid as stretchers; this is sometimes called "rowlock" bond. In the second type, the brick are laid part as stretchers and part as headers, as in ordinary brick-wall construction, with radial joints in which the outer end of the joint is thickened by increasing the thickness of the mortar or by insertion of thin pieces of slate. In the third method, the masonry is divided into blocks or sections (Figs. 122*b*, 128*e* and 130*d*).

Several forms of *vitrified-clay blocks* have been used for the construction of sewer arches as well as the entire sewer section. These blocks are of two general types, those intended to be laid in a single ring and those designed to be laid in a double ring. The sides of the blocks have tongue-and-groove joints, and the ends have lap joints. The blocks may be obtained for circular sewers in sizes from 30 to 108 in. in diameter and for egg-shaped sewers over a more limited range of sizes. Sewers of this form of construction, like the circular and egg-shaped brick sewers, depend upon the passive resistance of the backfill at the haunches for stability, unless concrete masonry or other adequate backing is provided. The smaller number of mortar joints as compared with brickwork is an advantage if these joints can be made substantially watertight, a process which has been found difficult in some cases. The hard, smooth, interior face of the blocks furnishes a good wearing surface for the invert; and if the blocks are of correct shape and well laid, the carrying capacity of the sewer should be as great as that of a well-built brick sewer.

Unreinforced concrete arches have been used to a considerable extent in recent years and have an advantage over stone or brick masonry arches in that the structure is more homogeneous and may withstand tensile stresses to a slight degree, although the arches should not be designed with this in view, and if cracks develop, they are likely to be larger and farther apart than in brickwork. In the design of unrein-

forced concrete arches, as well as those of stone and brick, the line of resistance should be made to fall within the middle third of the section, in order that no tensile stresses may be developed. If all the loads to act on the sewer were known exactly, it would be possible to design the section so that the line of resistance would lie within the middle third at all points. The loads caused by the action of earth pressure and live loads can only be approximated, however, and under special conditions the stresses in the arch section may not be entirely due to direct compression, but, in addition, bending stresses may be developed.

Arches of *reinforced concrete* are not subject to the limitations just mentioned but can be made to withstand heavy bending moments by reinforcing the section with steel bars to carry tensile stresses. In arches in which the line of resistance lies within the middle third, the stresses in the arch are mainly due to compression, and the concrete must of necessity carry the principal part of the load, in which case the steel will not be stressed to the allowable limit. The presence of the steel reinforcement is of considerable value, however, even in such cases. The steel furnishes an insurance to the structure, to care for tensile stresses which may occur on account of unequal settlement of the foundations, temperature changes, and many other conditions, of which the designer can have no exact knowledge. The steel affords an additional factor of safety against careless and defective construction. On account of its presence, it is possible to increase slightly the allowable working stresses in the concrete over those which should be used for unreinforced concrete masonry. Because of these considerations, the authors believe that, for large sewer arches, reinforced concrete offers greater advantages than unreinforced concrete, even though an analysis of the section may show that the line of resistance for the conditions considered will lie within the middle third of the masonry section. An inspection of the analyses given in Chap. XIV will show how great a change may occur in the theoretical location of the line of resistance due to a change in the assumed conditions.

Since cracks in reinforced concrete, which might permit moisture to reach the reinforcing steel, would be likely to result in corrosion, particular care should be taken to provide sufficient steel properly placed to ensure against such cracks.

A comparison of alternate types of sewers should be made on a basis as comparable as possible, particularly if competitive bids are to be taken. Figure 141 shows typical circular sections of equal diameter designed with this point in mind, but even in these designs, it is doubtful if the sections have equal strength or equal carrying capacity. Nearly equal carrying capacity can be obtained by varying the sewer area to correspond with the variation in coefficient of roughness for the several types

of construction and variation, if any, in the hydraulic radius of the section.

The left half of each section represents the condition where the bottom of the trench is firm enough to permit shaping in order to save concrete; and the right half meets the condition requiring a flat-bottomed trench. In the case of the monolithic concrete sections, under suitable conditions the trench could be made narrower and the concrete below the springing line could be placed against the side of the trench.

Probably, the concrete used in the three other types need not be so rich as in the first two types.

Disintegration of Concrete.—Reference has already been made in Chap. XI, in the discussion on concrete pipe, to the possibility of disin-

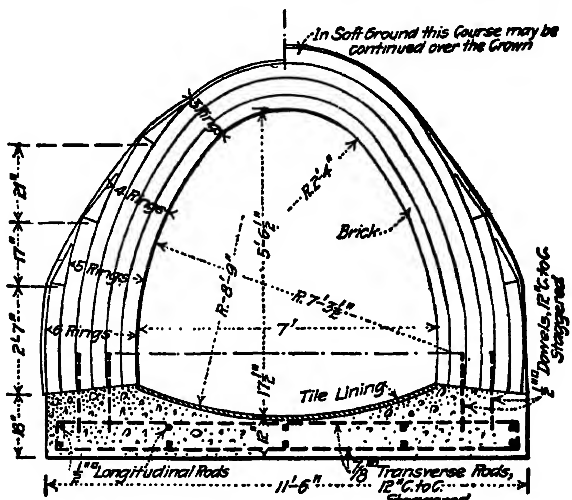


FIG. 142.—Semi elliptical sewer with brick arch, stockyards intercepter, Chicago.

tegrating action on concrete of seepage from acid or alkali soils. Where such destructive action is likely, consideration must be given to protective methods if concrete or cement mortar are to be used successfully.

Hydrogen sulphide given off by decomposing organic matter in sewage may form sulphuric acid above the flow line, which will attack some of the calcium compounds in the cement, forming soluble calcium sulphate. In forming, this compound expands and scales off the concrete and then gradually dissolves, leaving the aggregate uncemented. Thus the sewer may be seriously damaged. An interesting example of this is the Western Ave. sewer, Chicago, which was damaged by stockyards' sewage giving off hydrogen sulphide.[1] For a new sewer carrying similar sewage, the

[1] *Eng. News*, 1916; **76**, 486.

Sanitary District of Chicago adopted the brick-and-concrete design shown in Fig. 142, in preference to concrete, and at a somewhat higher cost, partly because of the protection from disintegration afforded by the brick arch and tile-lined invert.

Electrolysis in Concrete.—Considerable study has been given to the corrosive effect of stray electric currents in concrete reinforced with steel.[1]

In general, the danger of damage to reinforced concrete sewers by electrolysis is comparatively slight. Plain concrete is immune from such troubles. The trouble, if present, appears as cracking of the concrete around the steel bars at the point where the electric current enters the sewer. To have this condition, the electric current must flow between electrodes in contact with the concrete. The addition of salt to concrete (to prevent freezing during setting) and the presence of sea water or a salt marsh increases the susceptibility of reinforced concrete to troubles from electrolysis. If there appears to be any possibility of the sewer being exposed to the action of electric currents, the use of salt (calcium or sodium chloride) in concrete mixtures should be prohibited. Waterproofing the exterior of the concrete sewer will help as long as the membrane excludes moisture. Integral waterproofings appear to be of no avail. Where metal water or gas pipes, lead cable sheaths, and similar structures pass through reinforced concrete sewers (to be avoided, if possible), special care should be taken to prevent metallic contact with the steel reinforcing bars, and insulating material also should be used to prevent contact with the concrete.

Careful work on the part of electric-railway companies in maintenance of rail bonds and by taking other precautionary measures will help greatly in the prevention of electrolysis.

Wear on Sewer Inverts.[2]—In 1909–1910, a careful inspection by the authors of the condition of the brick sewers in Worcester, Mass., developed a number of interesting points. Many of these old brick sewers, forming a part of a combined system of sewerage and varying in size from a 24- by 36-in. to a 48- by 72-in. egg-shaped section, were constructed between 1867 and 1880. Natural or Rosendale cement was used in nearly every case, and the majority of the sewers were built by contract.

The brick invert was found to be badly worn in all sections where the velocity flowing two-thirds full exceeded 8 or 9 ft. per second, estimated by Kutter's formula with $n = 0.015$. In some sections where the

[1] For a discussion of this subject, the reader is referred to *Technologic Paper No.* 18, *Bur. Standards*, U. S. Department of Commerce, and to *Digest of Publications of Bur. Standards*, on "Electrolysis of Underground Structures Caused by the Disintegrating Action of Stray Electric Currents from Electric Railways," prepared by SAMUEL S. WYER, and published by the Bureau of Standards, January, 1918.

[2] See also, "Erosion of Sewer Inverts" in Chap. III.

estimated velocity amounted to 12 or 13 ft. per second, the first course of brick in the invert was worn through in places, and the second course was partly worn. A majority of the streets were surfaced with gravel, and during storms a large amount of street detritus washed into the sewers in spite of the many catchbasins. The effect of the scouring action of this material as it is swept or rolled along by the sewage can be seen on the bricks, which, especially below the dry-weather flow line, were worn to smooth faces and rounded edges.

On grades where the wear had been excessive, it was quite generally true that the upstream ends of the bricks were worn away more than the downstream ends. Figures 143 to 146 show bricks from sewers at Worcester. The two in Fig. 143 were taken from a 30- by 45-in. egg-shaped section built by contract in 1874. The masonry of this

Fig. 143.—Brick from arch and invert of Worcester sewer.

sewer was constructed of two rings of sand-struck brick of 20 to 30 per cent absorption, by volume, laid in Rosendale-cement mortar. The bricks shown were taken from a section where the grade is 0.0694. The velocity in this section (based on Kutter's formula, $n = 0.015$), at two-thirds full, is 22 ft. per second. The left brick was taken from the crown of the arch, on which there was no wear. The right brick was taken from the invert, the small end being the upstream end. The depth to which the mortar joints were washed out can be seen on the worn brick by the change in shade from dark to light, the light shade being caused by part of the mortar joint sticking to the brick. The mortar itself was very sandy and comparatively soft, and little difficulty was experienced in removing the brick from the invert.

Figure 144 shows two bricks taken from a 48- by 72-in. egg-shaped section, built in 1872 by contract, of 8-in. brickwork laid in Rosendale-cement mortar. These two bricks were taken from the side of the invert on a section where the grade was 0.0290 and the estimated velocity flowing two-thirds full was 20 ft. per second. The bricks were exceed-

ingly hard and dense, probably having an absorption of 8 to 12 per cent, and were worn very smooth, almost to a polish. The small end in each case was the upstream end. The bricks in the center of the invert were worn very much more than those shown, but owing to their excessive wear and consequent thinness and also on account of the depth

FIG. 144.—Brick from side of invert of Worcester sewer.

of sewage, it was impracticable to remove any of them. In this section, some of the first, or inner course, bricks, were worn through, and the second or outside course was beginning to show wear.

Figure 145 shows another brick taken from the same sewer and section as those shown in Fig. 143. This brick was laid in the invert

FIG. 145.—Brick from invert of Worcester sewer.

in the position shown in the photograph. The right end was the upstream end. There was a bad hole in the invert at this point, and the mortar was so completely washed out that the brick was removed with the fingers without the aid of a chisel. All that is left of one of the 4- by 8-in. faces is the little dark spot shown in the foreground at the

left-hand end. The brick was somewhat below the average in quality and rather porous.

Figure 146 shows a brick taken from the ledge, or step, above the invert in a manhole constructed in 1868 by contract. The brickwork was laid in Rosendale cement mortar. In this manhole there were five inlet pipes which discharged surface water from several catchbasins and inlets; they were so located that in time of storm the flow from all five was concentrated in a 4- or 5-ft. drop to the brick ledge of the manhole. The force of the falling water and detritus wore a bowl-shaped depression in the ledge and side of the manhole. The left end of this brick shows its original thickness, being protected by the brick in the course above. This brick shows more clearly than can be described the effect of the wearing action during a period of about 35 years. The next

Fig. 146.—Brick forming manhole ledge.

two bricks adjacent to the one shown were worn even more and broke in pieces in removal, owing to their extreme thinness. While this brick was not taken from a sewer invert, it shows very clearly the effect of even a small drop in the flow line and the resulting wear on the brickwork, such as might be expected from similar conditions in the invert.

The mortar joints were eroded to a much greater extent than the brick, which doubtless served to increase the wear on the brick, owing to the eddy currents caused by the additional roughness. This was not entirely due to the use of Rosendale cement, for the joints in the arch above the springing line were found to be in very good condition. Doubtless, some of the wear on the invert brick has been due to chipping action rather than abrasion.

In all cases where lateral sewers on steep grades entered well above the invert, there were signs of considerable wear on the side of the main sewer where the stream from the lateral struck during times of storm flow. In drop manholes and other places where a fall of 4 or 5 ft. or more occurred, the brickwork under the drop was badly worn.

On curves constructed on grades producing velocities of 8 ft. per second or more, the brickwork on the inside of the curve was cut away, in

several cases even through the second course of brick. A cross-section (Fig. 147*a*) of the interior surface of an egg-shaped section, 48 by 72 in., constructed by contract in 1872 of two rings of brick with Rosendale cement mortar on a grade of 4.32 ft. per 100 ft., shows this abrasion of the invert on a curve. Figure 147*b* is a cross-section of the same sewer on a straight section. In each of the deep holes shown, the first course of brick had been worn away, and part of the second. The dotted lines show the approximate original surface of the brickwork. These cross-sections were made by a specially constructed pantograph. Soft bricks were worn much more than hard bricks, but where one soft brick was surrounded by hard bricks, even these were worn more than a similar section where the bricks were all hard.

On flatter sections of 100 to 200 ft. in length on either side of which were steep sections, there was some wear, due, no doubt, to the fact that the velocity in the flat section, although not greater than 5 or 6 ft. per second theoretically, actually was much higher on account of the influence of the steeper sections above and below.

It is interesting to compare the experience gained at Worcester with information obtained at Louisville, from an inspection of old brick sewers. Where the velocity was high, there was but little wear of the brick, while, at Worcester, sewers having apparently the same velocity showed serious wear. The explanation is that, at Worcester, the street detritus contained a large quantity of quartz sand coming from streets which for many years were surfaced with gravel. There were also large deposits of sand and gravel in the city, and the soil as a whole contained a large amount of quartz. In spite of the large number of catchbasins in use, considerable quantities of sand and gravel entered the sewers, and as the detritus was carried along by the flow of sewage, the invert bricks were worn by the harder material. At Louisville, the soil is composed of clay and disintegrated limestone, and the streets were surfaced with crushed limestone, which, for the most part, is softer than the sewer brick. Even in sewers constructed of relatively soft brick, say those testing between 24 and 30 per cent absorption, there appeared to be but little wear from the velocities which at Worcester had caused serious wear. Although doubtless the detritus washed along the inverts at Louisville caused some wear, the attrition was much more effective upon the detritus itself than upon the sewer brick.

Figure 148 shows patterns of two plaster casts taken from the invert of one of the Northern Outfall sewers, middle level, 9- by 9-ft. section, leading to the Barking works, London, England. The upper pattern shows the eastern portion of the cast, and the lower pattern shows the western portion. The dotted lines show the approximate original outlines of the brickwork, and the approximate depth of wear can be judged by comparison with the thickness of the brick. The mortar joints are

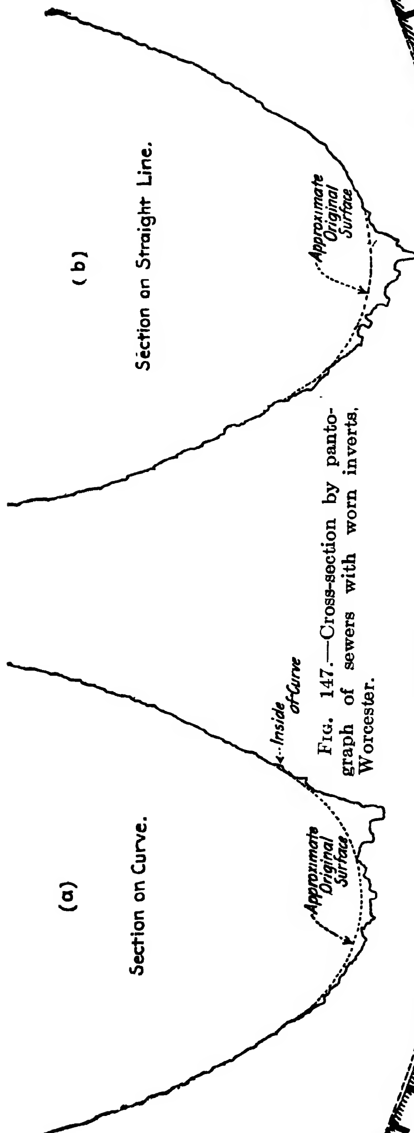


FIG. 147.—Cross-section by pantograph of sewers with worn inverts, Worcester.

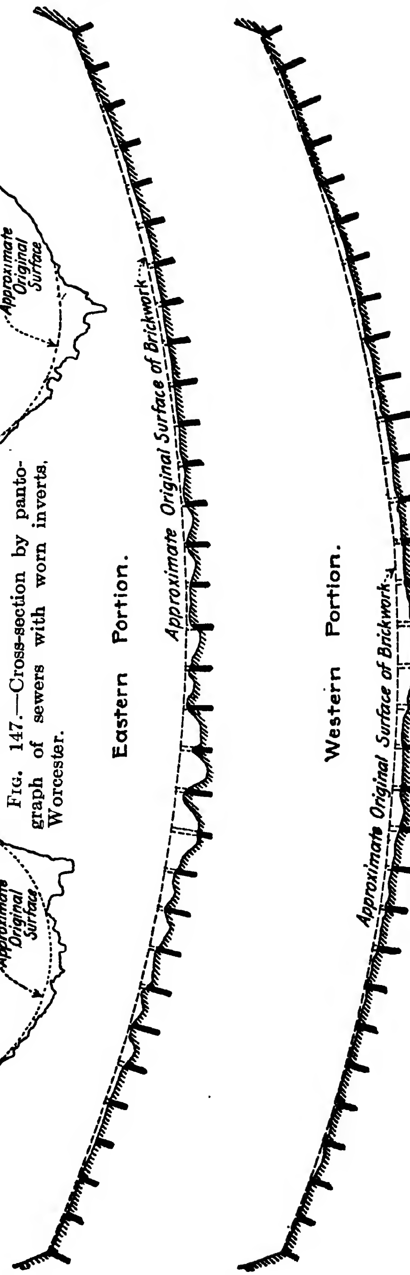


FIG. 148.—Patterns of plaster casts of inverts, London Northern outfall sewers.

indicated by heavy black lines. The most interesting feature of these patterns is that they clearly show that the cement mortar in the joints was harder than the bricks themselves and resisted the wear longer than the bricks did. This is exactly opposite to the experience in Worcester. This is the only instance which has come to the attention of the authors in which the mortar joints withstood the wear better than the bricks. Although the old sewers in Worcester were laid with Rosendale cement mortar, many of them have since been repaired with brick laid in portland-cement mortar, and in many cases even these new inverts have shown considerable wear. It is possible that, if these old sewers had been constructed in the first place with portland cement mortar, some such wear as that shown in Fig. 148 might have resulted.[1]

The full-size pattern from which Fig. 148 was made was furnished by John E. Worth, District Engineer of the London County Council. The casts were made Apr. 14, 1897. Mr. Worth states that the reported relative condition of the brick and mortar was so unusual that plaster casts of the invert were made in order to verify and preserve the record.

Lining for Concrete Sewers.—From the observations made and tests conducted by the authors, it appears that on all slopes in which the estimated velocity of the sewage will be 8 ft. per second or greater, the invert may well be paved with hard-burned or preferably vitrified paving brick with square edges, laid with the edges projecting as little as possible and with full portland cement mortar joints. This invert paving should extend well up on the sides of the sewer, on straight sewers covering, in general, the arc of an angle of 90 deg. at the center of a circular sewer. The use of paving brick, as above suggested, is preferable to concrete, on account of the greater ease of making repairs and, further, on account of the probability that vitrified or even hard-burned brick will withstand the wear better than concrete of average quality. It is desirable when sewers are to be built of concrete to use hard aggregates, especially for inverts, and a first-class granolithic finish where the surface is subject to greatest wear is better than the ordinary concrete finish.

The use of vitrified-clay liner plates in concrete sewers has been introduced where excessive erosion is anticipated or the concrete is likely to deteriorate from the effect of acid or alkali in the sewage. They have been used not only in monolithic concrete sewers but also in reinforced concrete pipe of large size. These tile liners are 9- by 18-in. plates with ¾-in. thickness and longitudinal dovetail ribs on the back to provide adequate bond with the concrete. They are made flat or with the face curved to a radius of from 16½ to 45 in. so as to be better suited to use in circular sewers. The curved plates are made in widths such that

[1] Observations similar to those made by the authors in Worcester are recorded in *Munic. Jour.* (1910) and also in *Eng. Rec.*, 1909; **60**, 629, by Edward S. Rankin, Engineer of Sewers and Drainage, Newark, N. J.

split plates are not required to construct the periphery of a sewer of given diameter. As such plates are difficult to cut to exact dimensions, it is advisable to procure sufficient plates of appropriate length to permit of carrying the lining around curves and of making closures without cutting.

There are two methods of laying the liner plates. They may be wired or otherwise attached to a skeleton form work, thus making the sheathing against which the concrete is run. This is the usual method employed for precast pipe, vertical walls, roofs, and arches. For floors and inverts, they are sometimes laid in cement mortar on the green concrete immediately after the removal of the forms. Satisfactory bond has been obtained by both methods. Irregularities in the tile make it impossible to obtain joints which are absolutely tight, so all joints should be pointed with cement mortar or other material.

To get the full advantage of tile liner plates, the joint should be as resistant to erosion and corrosion as the plates themselves. After three years of experimenting, the Los Angeles County Sanitation Districts (A. K. Warren, Chief Engineer) decided to use for jointing a compound of sulphur, sand, and ground silica in the proportions of 10 parts sulphur, 7 parts clean sand passing a 50-mesh sieve, and 3 parts finely ground silica, by weight. This mixture, which becomes fluid at 250° F. and crystallizes rapidly at a somewhat lower temperature, is run into the joints between the plates which have been attached to the forms, using special apparatus for keeping the compound hot, and applying it to the joint from the outside.[1]

[1] As described in *Eng. News-Record*, 1927; **99**, 346.

CHAPTER XIII

LOADS ON SEWERS

Notation.—The following is a list of notations used in this chapter:

B = breadth of ditch a little below top of pipe or sewer, or
= outside width of pipe or sewer
b = width of face of tamper
c = coefficient, in which allowance is made for ratio of depth to width of trench, for friction of backfill against sides of trench, and for character of backfilling material
F = height of fall of rammer for tamping backfill
f = compression of backfill under one blow of rammer at end of tamping
H = height of backfill above top of pipe
P = projection of culvert or pipe above ground surface, or
= total earth pressure by Rankine's formula
p = projection ratio = P/B
S = angle between direction of pressure and normal to surface acted upon
T = total concentrated load applied at a point on surface of embankment
V = average intensity of vertical pressure per square foot on top of pipe
V_p = intensity of vertical pressure per square foot on top of pipe due to concentrated load applied to surface of embankment
V_s = intensity of pressure at surface of backfill (as under a rammer)
W = weight of backfill on pipe, per linear foot
W_p = pressure on pipe per linear foot due to W_s
W_s = load applied per linear foot at surface of embankment, by surcharge or "superload"
w = weight per cubic foot of backfill
ϕ = angle of repose of material used as backfill

EXTERNAL PRESSURE ON PIPES

The external pressure or load carried by sewer pipes consists of that part of the weight of the earth above them which is transmitted to and

carried by the pipes, together with so much of the weight of objects upon the surface (such as trucks or piles of building material) as is similarly transmitted.

Two classes of conditions are to be considered: (1) where a trench has been dug for the pipe and the pressure results largely or wholly from the weight of the backfilling material which is confined between the comparatively firm walls of the trench and (2), where the pipe is laid on the surface of the ground, or in a trench of slight depth, and an embankment is then constructed over the pipe, in which case a much larger quantity of material contributes to the pressure on the pipe.

Frühling's Study.—One of the earliest attempts to estimate the pressure produced in a trench by backfilling was made by August Frühling.[1] He assumed that the vertical pressure due to backfilling increased at a diminishing rate as the depth increased, until, at a depth of 5 m., no further increase occurred. Further, he assumed that the total pressure at any depth varied according to a parabolic law. From these assumptions, he deduced the following formula:

$$V = w\left(\frac{D}{3} - \frac{(D - H)^3}{3D^2}\right)$$

where all units are metric and D is the depth below which there is no increase in V, taken as 5 m. If this expression is transformed to English measures and w is taken at 100 lb. per cubic foot, the formula becomes

$$V = 100H - 6.10H^2 + 0.124\,H^3$$

Barbour's Experiments.—Experiments by F. A. Barbour[2] were made by placing a modified hydraulic ram in the bottom of a 13-ft. trench and supporting a platform on the plunger. Sheeting was placed across the trench at each end of the platform, so as to confine the backfill placed on the latter. This series of experiments was not utilized in developing a formula, but the results were expressed in a number of curves. These give smaller pressures than those computed by the Frühling formula for depths less than about 15 ft., and greater pressures at greater depths.

Investigations by Marston and Anderson.—Marston and Anderson use in their analytical treatment of pressures in trenches[3] practically the same method that was developed by Janssen for the pressures in

[1] "Die Entwässerung der Städte."

[2] *Jour. Assoc. Eng. Soc.*, 1897; **19**, 193.

[3] The results of an elaborate investigation of the subject, lasting several years, were made public in 1913 in *Bull.* 31 of the Engineering Experiment Station of the Iowa State College of Agriculture. This was written by Prof. Anson Marston, director of the station, and A. C. Anderson, and contains the first well-developed comprehensive theory of the subject which was also checked by numerous experiments.

grain bins.[1] This gives for the weight on the pipe $W = cwB^2$, in which B is the width of the trench a little below the top of the pipe. The values of c are given in Table 132, and by the curves marked "Ditch Condition" (Fig. 153, p. 479).

TABLE 132.—APPROXIMATE SAFE WORKING VALUES OF c IN THE MARSTON AND ANDERSON TRENCH-PRESSURE FORMULA $W = cwB^2$

Ratio of depth to width[1]	Values of c for—			
	Damp top soil and dry and wet sand	Saturated top soil	Damp yellow clay	Saturated yellow clay
0.5	0.46	0.47	0.47	0.48
1.0	0.85	0.86	0.88	0.90
1.5	1.18	1.21	1.25	1.27
2.0	1.47	1.51	1.56	1.62
2.5	1.70	1.77	1.83	1.91
3.0	1.90	1.99	2.08	2.19
3.5	2.08	2.18	2.28	2.43
4.0	2.22	2.35	2.47	2.65
4.5	2.34	2.49	2.63	2.85
5.0	2.45	2.61	2.78	3.02
5.5	2.54	2.72	2.90	3.18
6.0	2.61	2.81	3.01	3.32
6.5	2.68	2.89	3.11	3.44
7.0	2.73	2.95	3.19	3.55
7.5	2.78	3.01	3.27	3.65
8.0	2.82	3.06	3.33	3.74
8.5	2.85	3.10	3.39	3.82
9.0	2.88	3.14	3.44	3.89
9.5	2.90	3.18	3.48	3.96
10.0	2.92	3.20	3.52	4.01
11.0	2.95	3.25	3.58	4.11
12.0	2.97	3.28	3.63	4.19
13.0	2.99	3.31	3.67	4.25
14.0	3.00	3.33	3.70	4.30
15.0	3.01	3.34	3.72	4.34

[1] The depth of trench is to the top of the pipe.

The approximate averages of a large number of measurements of weights and frictional properties of different classes of backfilling are given in Table 133. Within the range of ordinary ditch-filling materials, it takes a large difference in the values of the friction coefficients to make a material difference in the weight carried by the pipe. Marston and

[1] KETCHUM, "Retaining Walls, Bins and Grain Elevators."

Anderson point out that the real difficulty in selecting the proper values from the table lies in deciding upon safe and reasonable allowances for the probable saturation of the materials under actual ditch conditions.

TABLE 133.—PROPERTIES OF DITCH-FILLING MATERIALS
Marston and Anderson

Material	Weight of filling, lb. per cu. ft.	Ratio of lateral to vertical earth pressures	Coefficient of friction against sides of trench	Coefficient of internal friction
Partly compacted damp top soil.......	90	0.33	0.50	0.53
Saturated top soil.....	110	0.37	0.40	0.47
Partly compacted damp yellow clay...	100	0.33	0.40	0.52
Saturated yellow clay..	130	0.37	0 30	0.47
Dry sand............	100	0.33	0.50	0.55
Wet sand...........	120	0.33	0.50	0.57

The approximate maximum loads on pipes in trenches of different widths and depths are given in Table 134. The investigations of Marston and Anderson have convinced them that a 12-in. pipe will have to carry the same load as an 18-in. pipe, if each is placed in the bottom of a 24-in. trench, other things being similar. When, for construction reasons, a wide trench is necessary, they have shown that, in firm soil, the load on the pipe can be greatly diminished by stopping the wide trench a few inches above the top of the pipe and then excavating the narrowest trench in which it is practicable to lay the pipe, making special enlargements for the bells, if necessary.

Their experiments to test the accuracy of the theory upon which this and their other tables were based were made by weighing the load on different lengths of pipes hung at different depths in trenches, from a system of levers ultimately ending on the platforms of scales. Particular care was taken to avoid all test conditions likely to cause uncertainty regarding the accuracy of the results; and where doubt arose, the tests were repeated, with or without modification, until uncertainty was eliminated.

In commenting on Table 134, Marston and Anderson point out that the lateral pressure of the filling materials against the sides of the trench develops a frictional resistance which carries part of the vertical pressure near the sides of the trench, so that for the same depth of backfill, as at the level of the top of the pipe, the vertical pressure of the filling mate-

TABLE 134.—APPROXIMATE MAXIMUM LOADS, IN POUNDS PER LINEAR FOOT, ON PIPE IN TRENCHES, IMPOSED BY COMMON FILLING MATERIALS
Marston and Anderson

Depth of fill above pipe	Breadth of ditch at top of pipe									
	1 ft.	2 ft.	3 ft.	4 ft.	5 ft.	1 ft.	2 ft.	3 ft.	4 ft.	5 ft.
	Partly compacted damp top soil; 90 lb. per cubic foot					Saturated top soil; 110 lb. per cubic foot				
2 ft.	130	310	490	670	830	170	380	600	820	1,020
4 ft.	200	530	880	1,230	1,580	260	670	1,090	1,510	1,950
6 ft.	230	690	1,190	1,700	2,230	310	870	1,500	2,140	2,780
8 ft.	250	800	1,430	2,120	2,790	340	1,030	1,830	2,660	3,510
10 ft.	260	880	1,640	2,450	3,290	350	1,150	2,100	3,120	4,150
	Dry sand; 100 lb. per cubic foot					[1] Saturated sand; 120 lb. per cubic foot				
2 ft.	150	340	550	740	930	180	410	650	890	1,110
4 ft.	220	590	970	1,360	1,750	270	710	1,170	1,640	2,100
6 ft.	260	760	1,320	1,890	2,480	310	910	1,590	2,270	2,970
8 ft.	280	890	1,590	2,350	3,100	340	1,070	1,910	2,820	3,720
10 ft.	290	980	1,820	2,720	3,650	350	1,180	2,180	3,260	4,380
12 ft.	300	1,040	2,000	3,050	4,150	360	1,250	2,400	3,650	4,980
14 ft	300	1,090	2,140	3,320	4,580	360	1,310	2,570	3,990	5,490
16 ft.	300	1,130	2,260	3,550	4,950	360	1,350	2,710	4,260	5,940
18 ft.	300	1,150	2,350	3,740	5,280	360	1,380	2,820	4,490	6,330
20 ft.	300	1,170	2,420	3,920	5,550	360	1,400	2,910	4,700	6,660
22 ft	300	1,180	2,480	4,060	5,800	360	1,420	2,980	4,880	6,960
24 ft.	300	1,190	2,540	4,180	6,030	360	1,430	3,050	5,010	7,230
26 ft.	300	1,200	2,570	4,290	6,210	360	1,440	3,090	5,150	7,460
28 ft.	300	1,200	2,600	4,370	6,390	360	1,440	3,120	5,240	7,670
30 ft.	300	1,200	2,630	4,450	6,530	360	1,440	3,150	5,340	7,830
	Partly compacted damp yellow clay; 100 lb. per cubic foot					[1] Saturated yellow clay; 130 lb. per cubic foot				
2 ft.	160	350	550	750	930	210	470	730	1,000	1,240
4 ft.	250	620	1,010	1,400	1,800	340	840	1,330	1,870	2,370
6 ft.	300	830	1,400	1,990	2,580	430	1,140	1,900	2,630	3,410
8 ft.	330	990	1,720	2,500	3,250	490	1,380	2,360	3,360	4,100
10 ft.	350	1,110	2,000	2,920	3,880	520	1,570	2,760	3,980	5,270
12 ft.	360	1,200	2,220	3,320	4,450	540	1,730	3,100	4,560	6,050
14 ft	370	1,280	2,410	3,650	4,950	560	1,850	3,410	5,050	6,760
16 ft.	370	1,330	2,570	3,950	5,400	570	1,940	3,660	5,510	7,440
18 ft.	380	1,380	2,710	4,210	5,810	570	2,020	3,880	5,930	8,060
20 ft.	380	1,410	2,830	4,450	6,180	580	2,090	4,070	6,280	8,610
22 ft.	380	1,430	2,920	4,640	6,500	580	2,140	4,240	6,610	9,130
24 ft.	380	1,450	3,000	4,820	6,800	580	2,180	4,380	6,910	9,590
26 ft.	380	1,470	3,060	4,980	7,080	580	2,210	4,500	7,160	10,010
28 ft.	380	1,480	3,120	5,100	7,310	580	2,240	4,610	7,380	10,430
30 ft.	380	1,490	3,170	5,230	7,530	580	2,260	4,700	7,590	10,780

[1] These two subtables contain the most important figures for practical use.

rials, they state, is much greater in the middle of the trench than at the sides. Moreover, the side-filling material between the pipe and the sides is more compressible than the pipe and therefore can carry very little of the load. Hence, the pipe must have sufficient strength to carry the weight of all the backfill above the level of the top of the pipe, except that supported by friction upon the sides of the trench. Imper-

fections in the side filling and tamping probably increase the applicability of the principle.

Most analytical discussion of the pressures in trenches has been based upon the assumption of vertical sides. In many cases, the sides of the trench slope outward from its bottom, a condition which was investigated both analytically and experimentally by Marston and Anderson. An arching action apparently takes place, they found, between the sides of the trench and points at the ends of the top quadrant of the pipe. Above the elevation of these 45-deg. points, the material along the sides settles less than that in the center of the trench. The investigations referred to led to the conclusion that in these wedge-shaped trenches the proper width to substitute for B in the formula $W = cwB^2$ and to use as the width of the trench in Table 134, is the width at the height of the 45-deg. points on the pipe circumference, just a little below the top of the pipe.

Effect of Sheeting.—If sheeting is left in the trench, but the rangers removed, the friction between the backfill and the sides of the trench manifestly is decreased and the load on the pipe increased. The Marston and Anderson experiments indicate that this increase is from 8 to 15 per cent, and the experiments by Barbour[1] confirm this conclusion. If the rangers are left in place, the load coming on the pipes would probably be about the same as in unsheeted trenches, both according to theory and according to experiments by Barbour.

Rankine's Theory.—The theory of earth pressures given by Rankine is explained in Vol. II, p. 271, in its application to the estimation of pressure on sheeting. It is most commonly used in computing pressures upon retaining walls and the like, but may be extended to the estimation of earth loads upon sewers, in which case the surface of the earth is usually assumed to be horizontal. The total earth pressure acting on a section of a sewer arch may be considered as composed of a vertical component equal to the weight of the column of earth above the section and a horizontal component which at any point cannot be greater than $(1 + \sin \phi)/(1 - \sin \phi)$ times the vertical pressure at the same point, nor less than $(1 - \sin \phi)/(1 + \sin \phi)$ times the vertical pressure, ϕ being the angle of repose. The former expression represents the passive resistance of the earth, while the latter represents the active pressure which, at least, probably is realized. If the angle ϕ is taken as 30 deg., which is a convenient figure to use and approximately represents average conditions, the above statement means that the horizontal pressure cannot be greater than three times the vertical pressure nor less than one-third of it. While it is recognized that a more logical course would be to use exact values for the angle of repose, or, better, the angle of internal friction, this is hardly justified for ordinary conditions because of the

[1] *Jour. Assoc. Eng. Soc.*, 1897; **19**, 193.

great uncertainty regarding the action of earth pressures and the variation in the character and condition of trench materials.

Terzaghi's Studies.—It should be mentioned that the methods of Rankine and Coulomb, for determining earth pressures, have been called into question by Dr. Charles Terzaghi in numerous articles, from about 1920 to date,[1] and he has suggested new theories for the behavior of earths based upon their elastic properties. "Slip" is held to be an incidental rather than an essential event, since an appreciable deformation or movement of the wall or arch is required before "slip" can occur. As the structure yields, the intensity of the pressure is decreased. For example, with cohesionless sand where, according to accepted theories,

$$P = \tfrac{1}{2}cwH^2$$

c is 0.42 before deformation of the structure, while, after such yielding begins, c ranges from 0.15 to 0.05 or less.

Terzaghi's method of determining earth pressures is based on tests for compressibility, permeability, and various other characteristics of the earth in question. The method of utilizing these data is not yet sufficiently formulated for general application but requires considerable understanding of the nature and behavior of various kinds of earths. It is to be hoped that the increasing amount of research that is being carried out in various quarters will present sufficient data on earths of all kinds to permit a more logical and accurate method of determining earth pressure for which provision should be made.

Mohr's Method of Determining Pressures.—A graphical method of determining earth pressures, devised by Professor Mohr in 1871 and founded on Rankine's theory, was described by Prof. G. F. Swain[2] as follows:

Let a horizontal line AH (Fig. 149) represent the surface of the earth. Draw HI perpendicular to AH, and of some convenient length, as 5 in., equivalent to 10 ft. on a scale of ½ in. to 1 ft. Lay off

$$HK = HI \tan^2 (45° - \tfrac{1}{2}\phi)$$

where ϕ = angle of repose. This will be recognized as equivalent to Rankine's formula for the intensity of earth pressure, with w, the unit weight of earth, omitted.

$$P = wH \tan^2 (45° - \tfrac{1}{2}\phi)$$

where P = the total earth pressure per unit length of wall or sewer, by Rankine's theory, and H = the depth of earth.

Having located point K, with KI as a diameter, describe a circle. Through I draw a line IV_1 parallel to the face of the wall or section of arch upon which the pressure of the earth acts. Draw V_1K through the

[1] *Eng. News-Record*, 1920; **85**, 632; 1925; **95**, 742, 796, 832, 874, 912, 987, 1026.

[2] *Jour. Franklin Inst.*, 1882; **114**, 241.

points V_1 and K on the circumference of the circle, and prolong it to meet the surface line AH. At this point of intersection A, draw AI, which gives the direction of the active pressure on plane 1. The distance HV_1 measures the magnitude of this pressure for the depth represented by HI. $(HV_1/HI)w$ is the intensity of active pressure per unit depth of earth on plane 1. The magnitude of HV_1 can be obtained by scaling the line HV_1. In a similar manner, the direction and amount of the active pressure on any other plane, as plane 2, can be found.

The amount of the maximum passive earth pressure is measured by HI for a depth of HV_1 (HI/HV_1 = intensity) for plane 1, or by HI for a

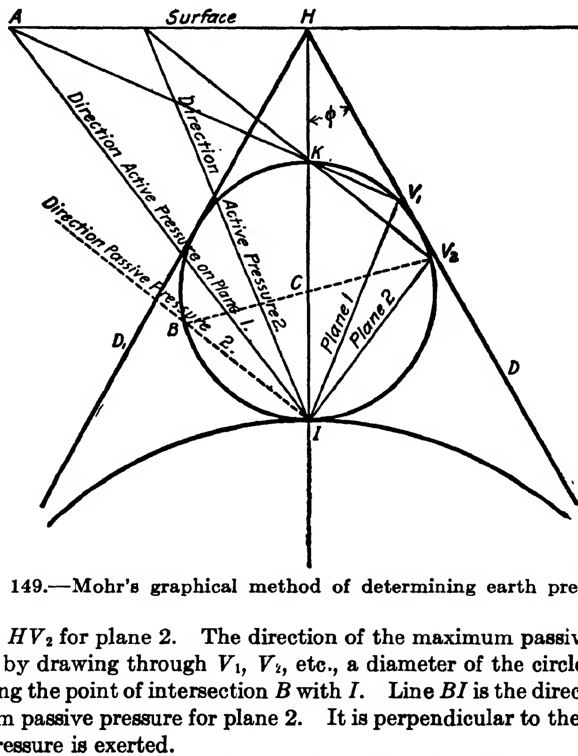


FIG. 149.—Mohr's graphical method of determining earth pressures.

depth of HV_2 for plane 2. The direction of the maximum passive pressure is found by drawing through V_1, V_2, etc., a diameter of the circle and then connecting the point of intersection B with I. Line BI is the direction of the maximum passive pressure for plane 2. It is perpendicular to the face upon which pressure is exerted.

There is an exact mathematical proof of the foregoing, but the following general proof will probably be sufficient:

If, in the figure we let the line HI represent a vertical plane, we have chosen HK of such a distance that for the depth HI, HK represents the intensity of the active earth pressure.

It can be proved that as the plane of the wall slants away from the vertical, a circle of diameter KI will contain all the points V for every position of the plane, the intensity being HV/HI until a horizontal surface is reached which has a pressure of $HV/HI = HI/HI = 1$, or the total dead

weight of the earth above the plane. The angle IHV is the angle S, or the angle which the direction in which pressure acts makes with the normal to the plane.

From Rankine's theory, we know that the angle S can never exceed the angle of friction ϕ, or the angle of repose of the earth. Hence, if we draw from H two lines making angles of ϕ on either side of HI, we know the circle must lie within those lines, and when the earth is just on the point of slipping, $S = \phi$ and the circle is tangent to the two lines HD and HD'. There are two circles which satisfy the conditions representing the two limiting states of equilibrium when the earth is just ready to slip. The larger circle, only part of which is shown in Fig. 149, represents the case where the maximum principal pressure HI is increased until the limiting condition is reached. This is the passive earth pressure. The smaller circle represents the case where the minimum principal pressure HK is decreased until the limiting condition is reached. This is the active earth pressure. In the case of $\phi = 30$ deg., for which the figure is drawn, the passive earth pressure is nine times the active. It is not necessary, however, to use the large circle, since for the active pressure

$$P_a = wH \frac{1 - \sin \phi}{1 + \sin \phi}$$

and for the passive pressure

$$P_p = wH \frac{1 + \sin \phi}{1 - \sin \phi}$$

the term $(1 - \sin \phi)/(1 + \sin \phi)$ being merely inverted. The inversion has been accomplished as follows:

The active pressure per unit depth $= w(HV/HI)$

The passive pressure per unit depth $= w(HI/HV)$

The angle IHV = the angle S, the angle between the normal to the plane and the direction in which pressure acts. Therefore, having this angle, we can erect a normal to the plane and lay off the angle S, thereby obtaining the direction of the stress. For example:

$$\text{angle } IAV_1 = \text{angle } IHV_1$$

Recent experiments (1926) at the Iowa Experiment Station[1] on the supporting strength of pipes in embankments "confirm the general correctness and reliability of Rankine's formula for calculating the active horizontal pressure in masses of granular materials."

SURFACE LOADS TRANSMITTED TO SEWERS

Live Loads.—Sewers constructed in shallow cut are often subjected to the effect of loads on the surface, transmitted through the earth filling. If the sewer line is crossed by steam-railroad tracks, there will be heavy loads from locomotives or loaded freight cars; if crossed by an electric railroad, there will be the loads from passenger or express cars, construction cars, or snow plows, which, in the case of high-class inter-

[1] *Bull.* 76, Engineering Experiment Station, Iowa State College, Ames, Iowa.

urban lines, amount to approximately the same as the loads on second-class steam railroads. In highways, sewers are subject to the loads of steam road rollers, traction engines, and heavy trucks.

TABLE 135

STANDARD LOCOMOTIVE LOADINGS.

Cooper[1]	Axle Spacing, Ft.	8'	5'	5'	5'	9'	5'	6'	5'		
Cooper[1] Class E-30	Axle Load	15,000	30,000	30,000	30,000	30,000	19,500	19,500	19,500	19,500	Uniform Load 3,000 Lb per Lin. Ft
Cooper[1] Class E-40	Axle Load	20,000	40,000	40,000	40,000	40,000	26,000	26,000	26,000	26,000	Uniform Load 4,000 Lb per Lin. Ft
Northern Pacific Ry[2]	Axle Load	25,000	52,000	52,000	52,000	52,000	36,000	36,000	36,000	36,000	Uniform Load 5,000 Lb per Lin. Ft
Atchison, Topeka & Santa Fe Ry[2] Heavy Grade	Axle Spacing in Feet	7.5'	4.5'	4.5'	4.5'	10.5'	5'	5.5'	5'		
Atchison, Topeka & Santa Fe Ry[2] Heavy Grade	Axle Load	27,000	66,000	66,000	66,000	66,000	30,000	30,000	33,000	33,000	Uniform Load 4,800 Lb per Lin. Ft

[1] From Cooper's General Specifications for Steel Railroad Bridges.
[2] From *Trans. Am. Soc. C. E.*, 1905; **54A,** 82.

TABLE 136

TYPICAL HEAVY FREIGHT CARS

Steel Coal Cars	Axle Spacing in Feet-Inches.	5'6"	19'9"	5'6"	
Steel Coal Cars	Axle Load	35,000	35,000	35,000	35,000
Iron Ore Cars	Axle Spacing in Feet-Inches	5'6"	17'9"	5'6"	
Iron Ore Cars	Axle Load	60,000	60,000	60,000	60,000

From *Trans. Am. Soc. C. E.*, 1905, **54A,** 85.

For convenience in estimating live loads, four tables are given: Table 135, typical standard locomotive axle loads and spacings; Table 136, typical axle loads of cars for heavy freight, such as coal or iron ore; Table 137, typical axle loads of the heavy type of electric cars for suburban service; and Table 138 the wheel loads and general dimensions

TABLE 137.—TYPICAL HEAVY ELECTRIC CARS

Long Island R.R. 1907 – 53 Tons [1]	Axle Spacing Feet	5.45'		5.5'	27.9'		6.7'		5.45'
	Overall, Feet	51.0'							
	Axle Load Pounds	30800		30800		22,200		22,200	
Boston Elevated Ry. Elevated Car No. 3 40.5 Tons [2]	Axle Spacing Feet		6.0'		26.25'		6.08'		
	Overall, Feet	46.60'							
	Axle Load Pounds	16,920		16,920		23,620		23,620	
Boston Elevated Ry. Cambridge Subway Car 62.9 Tons [2]	Axle Spacing Feet		6.0'		44.5'		7.0'		
	Overall, Feet	69.21'							
	Axle Load Pounds	28,000		28,000		34,900		34,900	
Interborough Rapid Transit Co. Manhattan Ry. Div. Car 38.0 Tons [2]	Axle Spacing Feet		5.0'		27.67'		6.0'		
	Overall, Feet	47.33'							
	Axle Load Pounds	15,850		15,850		22,150		22,150	
Interborough Rapid Transit Co. Subway Division Car 55.98 Tons [2]	Axle Spacing Feet		5.5'		29.92'		6.67'		
	Overall, Feet	51.33'							
	Axle Load Pounds	24,240		24,240		31,740		31,740	
Bay State St. Ry Co. Express Car 49.5 Tons [2]	Axle Spacing Feet		6.33'		15.67'		6.33'		
	Overall, Feet	42.0'							
	Axle Load Pounds	24,750		24,750		24,750		24,750	
Typical Passenger Car Trans Am Soc C.E. 1924; 87, 1277	Axle Spacing Feet		7.0'		27.0'		7.0'		
	Axle Load Pounds	30000		30000		30000		30000	
Typical Freight Car Trans Am Soc C E. 1924, 87, 1277	Axle Spacing Feet		6.0'		19.0'		6.0'		
	Axle Load Pounds	50000		50000		50000		50000	

[1] From Jour. Assn Eng. Soc. 1907; 43, 241 [2] From Proc Eng Soc West. Penn 1915, 31, 738 & 742

of steam road rollers, traction engines and heavy automobile trucks. Figure 150 shows the details of standard railroad track construction.

Locomotives have increased in weight to such an extent that most main-line railroad bridges are now designed for Cooper's Class E-50 loading, or heavier, which is five-fourths times the Class E-40 loading, shown in Table 134, on the same axle spacing.

Additional data regarding loads from electric cars, auto trucks, and occasional heavy loads to which highways are subjected can be found in a paper by Charles M. Spofford.[1]

In the *Second Progress Rept.* of the Special Committee to Report on Stresses in Railroad Track,[2] the results are given of tests of the distribution of pressure in the ballast under loaded railroad ties. For the standard track construction shown in Fig. 150, the vertical pressure at a depth of 30 in. below the underside of the ties was practically uniform for equal loads on the ties and in intensity amounted to 33 per cent of

[1] Highway Bridge Floors, *Proc. Eng. Soc. Western Penn.*, 1915; **31**, 727.
[2] *Trans. Am. Soc. C. E.*, 1919–1920; **83**, 1573.

TABLE 138.—WEIGHTS OF ROAD ROLLERS, TRACTORS, AND TRUCKS

Rating	Total weight equipped in pounds	Load per wheel in pounds	Diameter of wheels in inches		Face width of wheels in inches		Distance c. to c. of axles		Width of track, inches
			Front	Rear	Front	Rear	Ft.	In.	
Weights of steam road rollers (*Data furnished by the Buffalo Steam Roller Co.*)									
10 tons	26,000	8,670	44	69	47½	18	9	10	
12 tons	31,000	10,340	46	69	51	20	10	8	
15 tons	39,000	13,000	48	72	52½	22	11	1	94
Weight of traction engine (*Data furnished by the Good Roads Machinery Co.*)									
16 h.p.	19,580	6,530[1]	40	66	12	19	10	6	82
Weights of typical automobile trucks									
5 tons	20,000[2]	6,900[1]	36	42	6	13	12	6	86
20 tons[3]	40,000	16,000[1]	..	.	....	20	14	0	92

[1] Rear wheels.
[2] Allows for 25 per cent overload.
[3] *Trans. Am. Soc. C. E.*, 1924; 87, 1277.

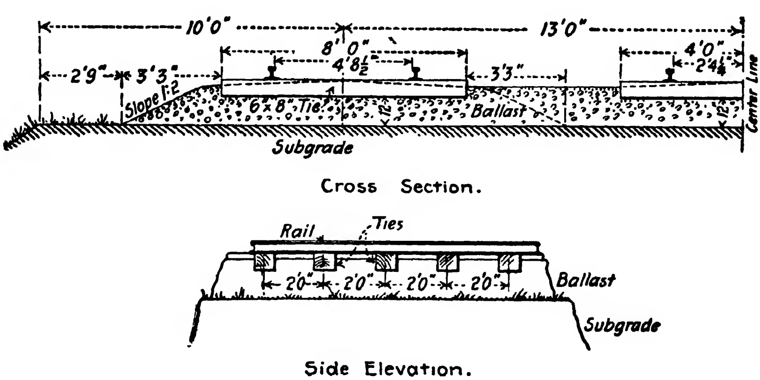


FIG. 150.—Standard railroad track construction.

the average intensity of pressure applied to the ballast by a tie (see Fig. 151). With ties spaced 18 in. instead of 24 in., c. to c., the pressure was uniform at a depth of 24 in. and amounted to 44 per cent.

From the results of tests of railroad-tie reactions,[1] it may be assumed that one axle load from locomotive drivers or car truck will be distributed over the least number of ties included in a distance equal to the axle spacing.

As an application of the above, if we assume a Cooper's Class *E*-40 locomotive loading on the standard track shown in Fig. 150, the average load on one tie from one driving axle spread over three ties may be taken as 13,333 lb., equivalent to 2,500 lb. per square foot of tie. At

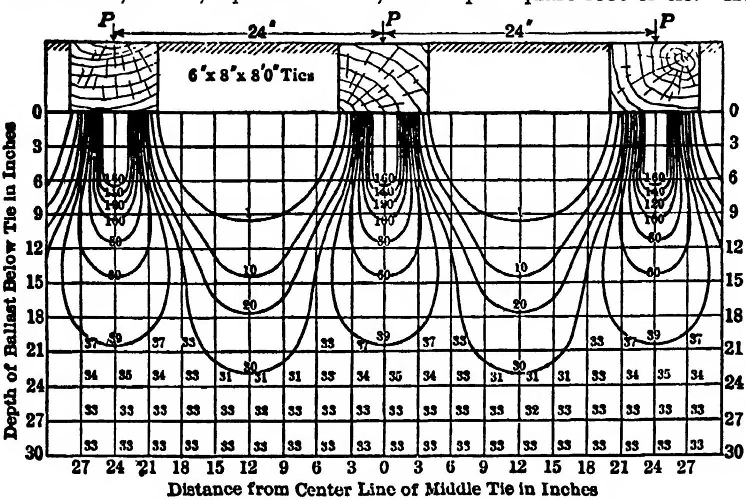


FIG. 151.—Lines of equal vertical unit pressure in ballast for equal loads on ties, in terms of percentages of the average unit pressure applied to the ballast by a tie.[2]

a depth of 12 in. in the ballast, according to Fig. 151, the average pressure may vary from about 77 per cent, or 1,930 lb. per square foot directly under the tie to 5 per cent, or 125 lb. per square foot, halfway between the ties. There is also some variation in intensity of pressure in a direction parallel with the ties. At a depth of 30 in. below the ties in the subgrade, the pressure can probably be assumed as substantially uniform and about 33 per cent of that at the tie, equivalent to 825 lb. per square foot.

If the railway track is built into grouted-granite block or other pavement resting on a concrete base, the pressures from wheel loads will be distributed over a greater area in less depth.

For locomotives the heaviest concentration would occur under the driving wheels, or, in the case of freight or passenger cars, under one truck. In Table 139 are estimates of the intensity of such loads.

[1] *First Progress Rept.* of the Special Committee to Report on Stresses in Railroad Track, *Trans. Am. Soc. C. E.*, 1918; **82**, 1269.

[2] *Trans. Am. Soc. C. E.*, 1919–1925; **88**, 1573.

TABLE 139.—ESTIMATED INTENSITIES OF SURFACE LOADS

Loading	Assumed pressure on underside of ties, assuming one axle load on the least number of ties between axles, pounds per square foot	Estimated equivalent intensity of load on a plane 30 in. below underside of ties, pounds per square foot
Locomotive, Cooper's Class *E*-30	1,875	620[1]
Locomotive, Cooper's Class *E*-40	2,500	825
Locomotive, Cooper's Class *E*-50	3,125	1,030
Locomotive, Northern Pacific R. R.	3,250	1,070
Locomotive, A. T. & S. F. R. R.	6,180	2,040
Steel coal car	2,190	720
Steel ore car	3,750	1,240
Electric car, Long Island R. R.	1,925	635
Electric car, Boston & Worcester St. Ry	1,560	515
Electric car, Boston Elev. Ry. Elevated Car No. 3	1,480	490
Electric car, Boston Elev. Ry., Cambridge Subway Car	2,120	700
Electric car, Interborough Rap. Tr. Co. Man. Ry. Div. Car	1,385	460
Electric car, Interborough Rap. Tr. Co. Subway Div. Car	1,985	655
Electric car, Bay State St. Ry. Co. Express Car	1,550	510
Electric car, Typical Passenger Car	1,875	620
Electric car, Typical Freight Car	3,125	1,030
	Estimated equivalent load on surface, pounds per linear foot of trench[2]	
Steam road roller 10 tons	8,670	
Steam road roller 12 tons	10,340	
Steam road roller 15 tons	13,000	
Traction engine 16 hp	6,530	
Automobile truck 5 tons (rating)	6,900	
Automobile truck 20 tons (total weight)	16,000	

[1] Assumed intensity of pressure 33 per cent of that on underside of ties.

[2] Assuming weight of one wheel per linear foot of trench. If trench is wide enough to

The loads from the wheels of steam road rollers, traction engines, trucks, etc., are applied directly to the surface of the fill but over a very small area. Although the intensity of the load at the surface is great, it becomes distributed fairly well over the entire width of the trench for a depth of 5 ft. or more and in a similar manner longitudinally.

Dead Loads.—In manufacturing districts, sewers are often subjected to heavy surface loads from piles of lumber, brick, pig iron, coal, etc. Wherever such is likely to be the case, ample allowance should be made. It is not uncommon to find surface loads as high as the following: lumber, 850 lb. per square foot; brick, 900 lb.; coal, 1,200 lb.; and pig iron, 2,300 lb.

There are cases, doubtless, where heavy masonry foundations have been built over sewers without regard for their stability. Wherever it is necessary to do such work, either the sewer arch should be strengthened to carry the excess load or, preferably, the foundation in question should be built so as to relieve the sewer arch of all of the load of the building or structure.

Proportion of Loads Transmitted to Sewers; Investigations of Marston and Anderson.—In order to determine the effect of such excess loads, Marston and Anderson[1] carried out an analytical and experimental investigation. They found that for a long load extending along the

TABLE 140.—PROPORTION OF "LONG" SUPERFICIAL LOADS ON BACK-FILLING WHICH REACHES THE PIPE IN TRENCHES WITH DIFFERENT RATIOS OF DEPTH TO WIDTH AT TOP OF PIPE

Marston and Anderson

Ratio of depth to width	Sand and damp top soil	Saturated top soil	Damp yellow clay	Saturated yellow clay
0.0	1.00	1.00	1.00	1.00
0.5	0.85	0 86	0 88	0.89
1.0	0.72	0.75	0.77	0.80
1.5	0.61	0.64	0 67	0.72
2.0	0.52	0.55	0 59	0.64
2.5	0.44	0.48	0.52	0.57
3.0	0.37	0 41	0 45	0.51
4.0	0.27	0.31	0.35	0.41
5.0	0.19	0.23	0.27	0.33
6.0	0.14	0.17	0 20	0.26
8.0	0.07	0.09	0.12	0.17
10.0	0.04	0.05	0.07	0.11

NOTE: Curves based on this table are given in Fig. 152.

[1] *Bull.* 31, Engineering Experiment Station, Iowa State College of Agriculture and Mechanic Arts, 1913.

trench, the decimal part of it which is transmitted to the pipe in trenches of different dimensions is approximately that given in Table 140.

Where the load is imposed by some short object, such as a road roller, the results of the investigation are not so positive, for it was found impracticable to test the theory upon which the analysis of such conditions was based. This theory was about the same as that found to be correct in other work when tested experimentally, so the results in this case are of considerable value even if purely theoretical. Apparently, the proportion of the load reaching the pipe depends on the ratio of the load along the trench: the width of the trench and on the ratio of the lateral and longitudinal pressures in the backfilling. The maximum and minimum values of the proportion of the load reaching the pipe are given in Table 141. The wide range of the figures in this table shows clearly that this particular portion of the investigation was not so directly applicable to practical problems as the rest of it.

TABLE 141.—PROPORTION OF "SHORT" SUPERFICIAL LOADS ON BACKFILLING WHICH REACHES THE PIPE IN TRENCHES WITH DIFFERENT RATIOS OF DEPTH TO WIDTH

Marston and Anderson

Ratio of depth to width	Sand and damp top soil		Saturated top soil		Damp yellow clay		Saturated yellow clay	
	Max.	Min.	Max.	Min.	Max.	Min.	Max.	Min.
0 0	1.00	1.00	1 00	1.00	1.00	1.00	1.00	1.00
0.5	0.77	0.12	0.78	0.13	0 79	0.13	0.81	0.13
1.0	0.59	0.02	0.61	0.02	0.63	0.02	0.66	0.02
1.5	0.46	....	0.48	....	0 51	....	0 54	
2.0	0.35	....	0 38	. . .	0.40	....	0.44	
2.5	0 27	....	0.29	....	0.32	....	0.35	
3.0	0.21	....	0.23	....	0.25	....	0.29	
4.0	0.12	....	0.14	....	0.16	....	0 19	
5.0	0.07	....	0.09	....	0.10	....	0.13	
6.0	0.04	....	0.05	....	0.06	....	0.08	
8.0	0.02	....	0.02	....	0.03	....	0.04	
10.0	0.01	....	0.01	....	0.01	....	0.02	

NOTE: Curves based on this table are given in Fig. 152.

The maximum proportions in Table 141 were found to occur when the length of the superficial load along the trench was equal to the trench width and the longitudinal earth pressure in the trench was one-half the lateral pressure. The minimum values occurred with the length of load equal to one-tenth the trench width and the longitudinal and lateral earth pressures equal.

In Fig. 152 are plotted curves of the values of c in the formula

$$W_p = cW_s$$

By "long loads" are meant those which extend a long distance along the trench as compared with its width and height. In this class come such loads as those resulting from piles of brick, lumber, pig iron, coal, etc., and, possibly, in freight yards, from long lines of cars on storage tracks.

By "short loads" are meant loads such as those from road rollers, trucks, or wagons and, in general, all of the other "live" loads previously mentioned.

The curves in Fig. 152 will be found of value in estimating the proportion of the weight of surface loads that might be transmitted through the backfilling to the sewer. All such loads, after having been reduced in the proportion shown by the curves or as aided by judgment, should

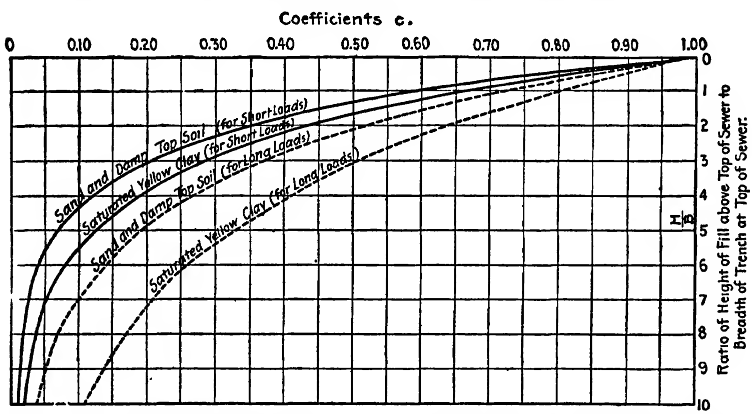


FIG. 152.—Coefficients of surface loads transmitted through earth fill in trench to sewers.

be added to the load due to the backfill obtained by the method previously described. In this way the backfilling and surface loading will be reduced to the load in pounds per linear foot on the pipe.

The allowance for increase in the load on the pipe due to impact in the case of the faster moving short loads will depend largely on the judgment of the engineer. It is believed, however, that for ordinary highway conditions, an increase of 50 to 100 per cent over the calculated load will be sufficient.

For example, the depth of earth over the crown of a certain sewer is 20 ft., and the width of trench at the top of the sewer is 10 ft. The backfilling material is sand weighing 120 lb. per cubic foot. One section of this sewer is to be built under a coal yard, and accordingly there should be added a surface load due to piles of coal of 1,200 lb. per square foot. The total

"long load" per linear foot of trench, W_s, would be 1,200 $\times$ 10 = 12,000 lb. The ratio of height of fill to width of trench, $H/B = 2$. On Fig. 152, follow along the horizontal line $H/B = 2$ until it intersects the curve for sand and damp top soil for long loads, which point is on the vertical line (interpolated) for coefficient $c = 0.52$. Substituting in the formula $W_p = cW_s$ the values of $c = 0.52$ and $W_s = 12{,}000$, we have $cW_s = 0.52 \times 12{,}000 = 6{,}240$ lb. per linear foot of sewer.

In order to determine the total load on the pipe it is necessary to obtain the load due to the backfill. The value of H/B is 2 as previously, and from Table 132, c in the formula $W = cw\ B^2$ would be 1.47. Then $W = 1.47 \times 120 \times 10^2 = 17{,}620$ lb. per linear foot due to the backfill. The total load on the pipe will be $17{,}620 + 6{,}240 = 23{,}860$ lb. per linear foot.

Other Experiments.—A. T. Goldbeck[1] and Prof. M. L. Enger[2] have also made experiments which are of interest in this connection, although these experiments are not extensive enough to permit of direct application to sewer design.

Possible Load on Pipe Resulting from Tamping Backfill.—An example of the possible use of Table 141 is given by Marston and Anderson in a discussion of the probable correctness of the general impression that pipes with a small depth of cover are susceptible to greater damage than those in deep trenches and that more damage is done during tamping than is frequently considered probable by those who write specifications for pipe. The maximum pressure V_s on the backfill, resulting from the shock of a blow of a rammer, is $2TF/f$, where T is the weight of the rammer in pounds.

The data for an example of the use of the formula may be taken from a discussion by J. N. Hazlehurst. Here the original "very thorough" tamping was done with a 40-lb. rammer on a 6-in. clay cover, resulting in some cracking of the pipe, while, later, the use of a 30-lb. rammer on a 12-in. fill had no such result. If it be assumed that very thorough tamping on a 6-in. cover is such as would produce a final compression f of 0.01 ft. under one blow, and the height of fall F was 0.5 ft., then, with a 40-lb. rammer, $V_s = 4{,}000$ lb. If the rammer had a face width b, of 0.67 ft., then the ratio of depth of cover to the width over which the load was applied, $H/b = 0.5/0.67$, was 0.75. The percentage of V_s reaching the pipe would be, from Table 141, about 71. Hence, about 2,800 lb. would be directly transmitted to an 8- by 8-in. area of pipe. With the lighter rammer, f would probably be a little larger, say 0.015 ft., because the cover was 1 ft. instead of 0.5 ft. The same method of computation as in the first case shows that the pressure on the 8- by 8-in. area would be about 1,000 lb. The correctness of the opinion occasionally expressed, that the use of a rather thick cover and light rammer in the lower part

[1] *Proc. A. S. T. M.*, 1917; **17**, Part II, 641.
[2] *Eng. Record*, 1916; **73**, 106.

of the trench is desirable for the safety of the pipe, is confirmed by this analytical method of investigation.

PRESSURE IN EMBANKMENTS

Marston's Culvert-pipe Investigations.—Beginning in 1915, Prof. Anson Marston has carried out an extensive investigation of the loads transmitted to culvert pipe, meaning thereby pipe which were laid on or slightly below the natural surface of the ground and then covered by an embankment. This is the usual condition experienced in building culverts. It is not uncommon in sewer work, particularly for outfall or intercepting sewers, or when the sewers are constructed at the same time as highway embankments.

The experiments were made in a similar way to those on pressure in trenches. It was found[1] that the same expression as for pressure in trenches was applicable, namely,

$$W = cwB^2$$

in which B is the greatest outside width of the pipe or culvert, while c is considerably greater than for pipes in trenches.

The tests showed that if the material of the embankment was consolidated by wetting and rolling, to an elevation above that of the top of the culvert amounting to at least one-third of the total height of the embankment above that point, and a trench were then excavated in the compacted earth and the culvert built, the resulting loads upon the pipe after the embankment was completed were the same as though the pipe had been laid entirely in a trench; in other words, "ditch conditions" prevailed, and the coefficients given in Table 132 were applicable. If the pipe were laid first, either on or somewhat below the natural surface, and the embankment built over it—called "projection condition," since a part of the pipe projects above the original surface—it was found that coefficients depending, in part, upon the extent of such projection were applicable, as already noted.

If the lower part of the embankment was constructed and consolidated before laying the pipe, but to a height above the top of the culvert less than one-third of the total height of the embankment above that level, then "imperfect ditch conditions" existed, and the values of the coefficient were uncertain but somewhere between those for "projection conditions" and "ditch conditions."

Figure 153[2] shows graphically the values of the coefficient c for ditch conditions, for projection conditions with various amounts of projection, and for the intermediate zone of "imperfect ditch conditions."

[1] *Second Progress Rept.* on Culvert Pipe Investigations, Iowa Eng. Experiment Station, 1922.

[2] Reproduced from MARSTON'S *Rept.*

The experiments were carried out with embankments of various heights, up to 20 ft. The applicability of the coefficients to conditions

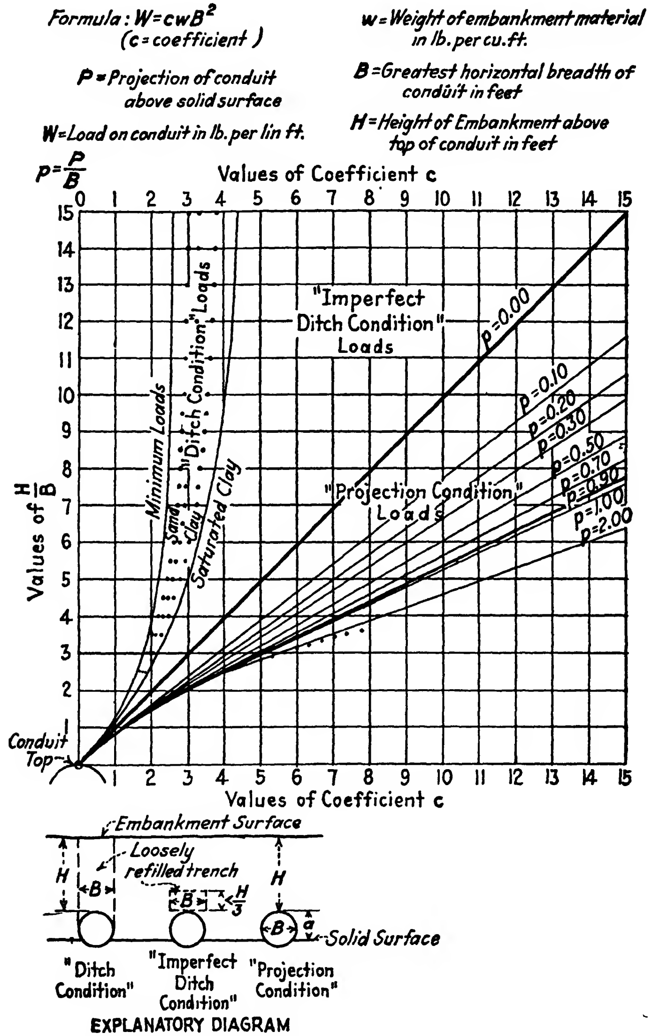


Fig. 153.—Computation diagram for loads on conduits. (*Marston.*)

in which the height of embankment materially exceeds 20 ft. is, therefore, open to question.

The horizontal pressure exerted by the material of the embankment is best determined by Rankine's formula given on p. 465.

WEIGHT OF BACKFILLING

For much designing work, it is sufficient to assume that the backfill will weigh 120 lb. per cubic foot and that the horizontal pressure at any depth due to this fill will be one-third of the vertical pressure. Where more precise figures are required, the material which will be used should actually be weighed, in a moist as well as a dry condition, or the information given in Table 133, p. 463, should be employed.

CHAPTER XIV

THE ANALYSIS OF STRESSES IN SEWER SECTIONS

Preliminary Design of Section.—The first step in the design of a sewer section, after selection of the shape of section to be used, is to make a preliminary design, based upon sections which have been used previously or with dimensions fixed by some empirical rule, as given in Chap. XII. The stresses to which the section may be subjected should then be analyzed and the design modified if necessary, so as to provide ample strength for the anticipated loads and at the same time be economical in material and construction costs.

Estimation of Loads.—The estimation of the external forces to which the sewer may be subjected is attended with quite as much uncertainty as any other step in the design. The dead weight of the structure itself and of the earth over it can be computed with a fair degree of accuracy; but whether there may be a future surcharge from additional fill or from storage of heavy materials, what temporary or moving loads may be applied, to what extent the sides of the trench may take a part of the load of the earth, or whether the sewer may have to carry more than the earth directly over it can only be approximated. The amount of the horizontal components of the thrusts from the earth can also be estimated only approximately. It is, however, necessary to assume the loads which the sewer is expected to carry, before its ability to sustain them can be computed; and the methods and data given in Chap. XIII will be of assistance in doing this.

Methods of Analyzing Stresses.—Three methods of analyzing the stresses in sewer sections are described in detail herein, and diagrams and computations illustrating the application of each of them to a horseshoe section 15 ft. 6 in. by 15 ft. 2 in. are given, based upon a preliminary design in which an arch 11 in. thick at the crown is assumed. This section appears to have sufficient strength if the character of the foundation is such that the entire load carried by the arch is transmitted to the earth or rock below the side walls; but if the sewer is to be built in compressible earth so that the entire section must be considered as an elastic ring, the load being transmitted to the earth across the whole of the invert, the unit stresses will be too high, and the section should be redesigned with thicker arch and invert.

The first method, herein called the "method of static analysis," is based on the so-called "hypothesis of least crown thrust" and is appli-

cable only to a limited portion of the arch and side walls. It assumes that the foundation is unyielding. This method is intended primarily for unreinforced masonry arches.

The second method, herein called "method of analysis according to the elastic theory," is based on the elastic theory of the arch and follows the method described by Turneaure and Maurer;[1] it is applicable to all sewer sections and loadings. Its application to conditions in which the entire section must be considered as an elastic ring is somewhat laborious because of the greater number of trial divisions involved.

The third method, also based on the elastic theory but using the so-called "method for indeterminate structures," is of special advantage in the analysis of the entire sewer section as it allows more freedom in subdividing the section. For large sewers, constructed in compressible soil and built of monolithic reinforced concrete, this method is the most advantageous.

Particular attention is called to the fact that in the following analyses the terms "elastic theory" and "method for indeterminate structures" are used as names for two methods both of which are based on the elastic theory and are applicable to indeterminate structures. The practical difference between the two is in the method of subdividing the sewer section for the analysis.

Since the three examples are based so far as practicable on the same assumptions, a direct comparison may be made of the results obtained, as is done hereinafter.

A fourth method which has sometimes been used, but which is not given herein, is that of Prof. Chas. E. Greene[2] for an arch rib with fixed ends. According to Horner, this method was used for the earlier arches designed under his direction. Later, it was worked up in the form of general formulas for each 10-deg. point on the arch. Similar formulas have been published by A. E. Lindau.[3] Horner stated that

> . . . this method is satisfactory where the arch rests on rock or on a heavy invert, but the introduction of a side wall of over a foot in height causes the whole structure to depart somewhat from fixture at the spring line.

He further states that Greene's method was used in 1914 to obtain a trial section for all larger arches or work of special importance and that the work was checked by the elastic-theory method of Turneaure and Maurer.

"Voussoirs" and "Joints."—In each of the methods of analysis, the arch ring (or the entire sewer section) is divided into segments by radial lines, *i.e.*, lines drawn normal to the center line of the ring. For the

[1] "Principles of Reinforced Concrete Construction."
[2] "Trusses and Arches," Part III, John Wiley & Sons, Inc. (1903).
[3] *Trans. Am. Soc. C. E.*, 1908; **61**, 387.

purpose of this discussion, the segments are called "voussoirs," and the lines between them are called "joints." It should not be inferred that these are joints in the masonry, as this, on the contrary, is supposed to be monolithic. The use of the term "voussoir," which means an arch stone, to include any part of the full section, is also somewhat anomalous; however, there do not appear to be any other words which can be used without ambiguity, to express the desired meanings.

"Skewback" and "Springing Line."—"The inclined or horizontal surface where the arch begins . . . is called the *skewback;* the inner edge of the skewback . . . is the *springing line.*"[1] There may be a difference between the plane where the arch begins and the one where it appears to begin. For instance, a semicircular arch appears to begin at the horizontal plane passing through the center. There are cases, however, in which the lower portions of the apparent arch are really curved upward extensions of the abutment walls, the real skewbacks being inclined planes higher than the center of the arch.

ANALYSIS OF STRESSES BY "STATIC" METHOD

There are a number of theories on which the static analysis of stresses in arches has been based, but the one most generally employed is the rational theory, based on the hypothesis of least crown thrust. The following application of this theory to the design of sewer arches is based on a discussion by Baker.[2]

According to the hypothesis of least crown thrust, the true line of resistance of the arch is that for which the thrust at the crown is the least possible in amount consistent with the arch being in a state of equilibrium. This theory assumes that the external forces acting on the arch create a thrust at the crown sufficient in amount to establish equilibrium in the arch and that when this state of equilibrium has been established, there is no need for further increase in the amount of this thrust and, therefore, the thrust is the least possible consistent with equilibrium. These assumptions do not of themselves locate the line of resistance, but if the external forces are known in amount, position, and direction, and the direction and point of application of the thrusts are assumed, sufficient data will be provided to locate the line of resistance corresponding to the least possible crown thrust. The rational method assumes that the earth pressure acting on the arch is composed of vertical and horizontal components.

This method is not applicable to arches unsymmetrically loaded. Methods of analyzing arches with unsymmetrical loads are long and laborious, and it is best to provide a section of ample size with steel in both faces to allow for reversal of stresses.

[1] SWAIN, "Structural Engineering," iii, 402.

[2] "Treatise on Masonry Construction," Tenth Edition, 620.

It is desired to locate the line of resistance of the 15-ft. 6-in. span concrete arch shown in Fig. 154, the thickness of the arch having been assumed, either with the aid of one of the empirical formulas previously given or in the light of experience with arches already constructed. As it has been assumed that the arch is symmetrically loaded, only half of the section need be drawn, as shown in the figure. Assume that the arch supports a depth of earth of 24 ft. above the crown, that the unit weight of the earth is 100 lb. per cubic foot, and that the unit weight of the concrete masonry is 150 lb. per cubic foot.

Division of the Arch into Voussoirs.—The masonry is divided by "joints" spaced uniformly along the center line of the ring, as shown in Fig. 154, into a number of "voussoirs." The joints are taken approximately normal to the center line of the ring. This analysis is for the purpose of determining the location and magnitude of the forces within the masonry at the joints. In this and in further discussions, a 1-ft. length of arch ring is always assumed.

It is necessary to assume the voussoirs small enough so that no material error will result from the assumptions as to centers of gravity, points of application of forces, etc., and so that there is little likelihood of material increase in thrust at the crown if the true "joint of rupture" (defined below) lies between two of the assumed joints. Unnecessary increase in the number of joints assumed, however, is to be avoided because of the increased number of computations involved.

External Forces.—The external forces acting at any voussoir are the weight of the voussoir itself and the pressure resulting from the earth over it, together with any superimposed loading, which may be taken as equivalent to an additional earth fill. The weight of the voussoir may be considered as a vertical force acting at it's center of gravity; but for purposes of this analysis, its point of application is assumed to coincide with that of the vertical earth load on the voussoir. This involves only a small error, for the weight of the voussoir is usually small compared with the vertical earth load. The pressure of the earth results from its weight, but its direction may be either vertical or inclined. In the following illustrative example we assume that the resultant vertical component of the earth pressure is equivalent to the weight of the prism of earth directly above the extrados of the voussoir, acting through the center of the horizontal projection of the extrados. The horizontal component of the earth pressure may be estimated by the methods described in Chap. XIII. Generally, it may be assumed that the intensity of the horizontal pressure is one-third of the vertical and that the resultant horizontal component acts through the center of the vertical projection of the extrados. (This point is near enough to the center of pressure for practical purposes, in most cases, particularly in view of the uncertainty of the actual amount of the horizontal

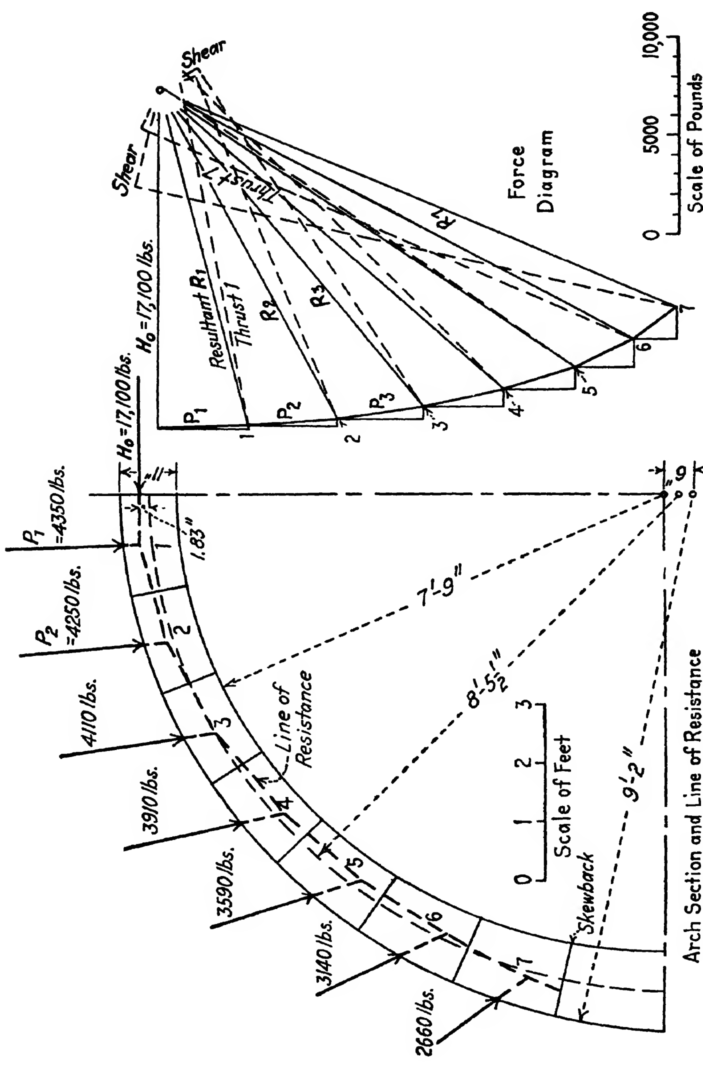


FIG. 154.—Analysis of semicircular arch by the "static" method.

pressure.) The resulting horizontal and vertical forces for each voussoir are given in Table 142.

Caution: This method of computing the load was used in the first edition of this book and has not been changed because the examples still illustrate the methods of analysis. The above loading would occur in the case of a trench wider than the masonry and a backfill of a purely granular nature, there being no assistance due to friction on the sides of the trench and no arching action in the material. It is the recommendation of the authors

TABLE 142.—COMPUTATIONS OF EXTERNAL FORCES

1 Voussoir number	2 Weight of concrete, pounds	3 Vertical earth pressure, pounds	4 Total vertical force, pounds	5 Horizontal earth pressure, pounds	6 Resultant force on voussoir, pounds
1	230	4,120	4,350	140	4,350
2	230	4,000	4,230	390	4,250
3	240	3,820	4,060	650	4,110
4	250	3,540	3,790	920	3,910
5	270	3,120	3,390	1,170	3,590
6	290	2,510	2,800	1,430	3,140
7	310	1,780	2,090	1,650	2,660
8	330	890	1,220	1,850	2,220

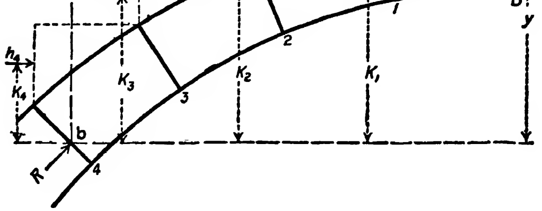


FIG. 155.

that, for ordinary conditions where the sewer is constructed in a trench, the Marston and Anderson method, discussed in Chap. XIII, be used to obtain the load per linear foot. This load should then be assumed to act on the top quadrant of the section. The horizontal load to be assumed on the voussoirs will depend on the amount of saturation expected, the extent of compacting the backfill on the sides of the section, etc., and is distinctly a matter of judgment on the part of the engineer. The most severe condition

usually results when the vertical load is distributed over the top quadrant and little or no horizontal load is assumed along the sides of the section.

Crown Thrust.—The portion of the arch shown in Fig. 155 is held in equilibrium by the vertical and horizontal forces due to the sum of the weights of concrete and earth prism and the horizontal component of the earth pressure, by the reaction R at the springing line or abutment, and by the thrust H at the crown. The direction of the reaction at the abutment is immaterial in this discussion.

Let H = thrust at the crown

H_0 = minimum crown thrust required to maintain equilibrium

H_1 = crown thrust taking moments about the inner third points of the joints

H_2 = crown thrust taking moments about the outer third points of the joints

b, b_1, b_2, etc. = origin of moments or point of application of reaction R at any joint

x_1, x_2, etc. = horizontal distance from point of application of reaction R on any joint under consideration, to the line of action of w_1, w_2, etc.

w_1, w_2, etc. = total vertical forces on the successive voussoirs, as shown in Fig. 155

K_1, K_2, etc. = vertical distance from point of application of reaction R to the line of action of h_1, h_2, etc.

h_1, h_2, etc. = horizontal components of forces acting on the successive voussoirs

y = vertical distance from point of application of reaction R to the line of action of H

Then, by taking moments about the point of application of the reaction, we have the following equation:

$$Hy = w_1x_1 + w_2x_2 \cdot \cdot \cdot + w_nx_n + h_1K_1 + h_2K_2 \cdot \cdot \cdot + h_nK_n.$$

From this we obtain

$$H = \frac{\Sigma wx + \Sigma hK}{y}$$

From the above equations it appears that, other things remaining the same, as y increases, H decreases, and, therefore, in order to obtain a minimum value of the thrust H, its point of application should be as near the extrados as is possible without exerting too great stress upon the masonry. It is usually assumed that the thrust acts at a point one-third the depth of the arch from the extrados at the crown. This assumption means that the unit compressive stress at the crown is equal to twice the thrust H divided by the thickness of the arch at that point, and the minimum compressive stress is 0.

It is also usually assumed that the thrust acts horizontally. If the arch is symmetrically loaded, this assumption is a reasonable one, but for conditions where the arch is unsymmetrically loaded, the thrust at the crown cannot be horizontal, and, on that account, a direct determination of the line of resistance by this method is impossible.

Table 143.—Computations of Moments about Inner or Lower End of Middle Third, and Crown Thrusts

Origin of moments at joint	Arms of vertical forces in feet								Arms of horizontal forces in feet								y feet	Σwx foot-pounds	ΣhK foot-pounds	H_1 total crown thrust $\frac{\Sigma wx + \Sigma hK}{y}$ pounds
	x_1	x_2	x_3	x_4	x_5	x_6	x_7	x_8	K_1	K_2	K_3	K_4	K_5	K_6	K_7	K_8				
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)	(21)
1	0.74								0.69								0.47	3,220	100	7,060
2	2.26	0.58							1.15	0.82							0.92	12,300	489	13,900
3	3.66	1.98	0.39						1.88	1.57	0.95						1.67	25,900	1,490	16,400
4	4.90	3.22	1.63	0.19					2.89	2.58	1.96	1.05					2.68	42,200	3,660	17,100
5	5.91	4.23	2.64	1.20	−0.03				4.12	3.81	3.19	2.28	1.10				3.89	58,800	7,510	17,000
6	6.69	5.01	3.42	1.98	0.75	−0.25			5.51	5.20	4.58	3.67	2.49	1.09			5.29	73,500	13,600	16,500
7	7.17	5.49	3.90	2.46	1.23	0.23	−0.48		7.02	6.71	6.09	5.18	4.00	2.60	1.02		6.80	83,300	22,400	15,500
8	7.37	5.69	4.10	2.66	1.43	0.43	−0.28	−0.61	8.57	8.26	7.64	6.73	5.56	4.16	2.58	0.87	8.36	87,400	33,900	14,500

Table 144.—Computations of Moments about Outer or Upper End of Middle Third, and Crown Thrusts

Origin of moments at joint	Arms of vertical forces in feet								Arms of horizontal forces in feet								y feet	Σwx foot-pounds	ΣhK foot-pounds	H_2 total crown thrust $\frac{\Sigma wx + \Sigma hk}{y}$ pounds
	x_1	x_2	x_3	x_4	x_5	x_6	x_7	x_8	K_1	K_2	K_3	K_4	K_5	K_6	K_7	K_8				
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)	(21)
1	0.78								0.38								0.15	3,390	53	22,950
2	2.38	0.68							0.74	0.51							0.63	13,230	300	21,500
3	3.83	2.14	0.55						1.60	1.29	0.65						1.40	27,990	1,150	20.800
4	5.14	3.45	1.87	0.40					2.63	2.30	1.68	0.78					2.41	46,070	3,080	20,400
5	6.22	4.53	2.94	1.50	0.22				3.90	3.57	2.93	2.03	0.87				3.68	64,580	6,725	19,400
6	7.04	5.35	3.78	2.31	1.05	0.03			5.35	5.03	4.40	3.50	2.30	0.92			5.12	81,040	12,800	18,350
7	7.56	5.88	4.30	2.82	1.57	0.55	−0.20		6.90	6.60	5.96	5.07	3.90	2.50	0.94		6.70	92,370	21,750	17,050
8	7.78	6.10	4.51	3.06	1.80	0.78	0.04	−0.4	8.56	8.24	7.60	6.70	5.53	4.14	2.58	0.88	8.34	97,420	33,780	15,740

Joint of Rupture.—The joint of rupture is that joint which requires the largest crown thrust H to prevent opening at its intradosal end. If the crown thrusts acting horizontally at the upper third of the crown joint, which are necessary to balance the moments of the external forces about the lower third points of the several joints, be computed, that joint which requires the greatest crown thrust is the joint of rupture, and the corresponding thrust is the thrust to be used in the construction of the line of resistance.

The extent of the arch which may be analyzed by this static method is from the crown down to the joint at which the line of resistance passes out of the middle third of the arch ring, this joint being, in fact, the skew-back of the arch. Below that joint, the ring is to be considered as a part of the abutment.

Crown Thrust for Joint Rupture.—The total vertical forces as computed are given in Table 142. The moment arm of each of these forces or weights, with reference to the several origins of moment, is measured and entered in Table 143. In the same table are columns headed Arms of Vertical Forces, and to the right of these, a series of columns headed Arms of Horizontal Forces. In column 2 will be found 0.74, which is the perpendicular distance or moment arm of the force, or weight, w_1 (Fig. 155) about the lower-middle-third point of joint 1. Similarly, 2.26 is the arm of w_1, about the lower-middle-third point of joint 2; 3.66, about the lower-middle-third point of joint 3; 4.90, about the lower-middle-third point of joint 4. In the same manner, 0.58 in column 3 is the arm of the weight w_2 about the origin of moments or lower-middle-third point of joint 2.

The horizontal forces as computed are given in Table 142. In a similar manner as described above, the arms of the horizontal forces, h_1, h_2, etc., are scaled and entered in Table 143. The moment arms of these horizontal forces denoted as K_1, K_2, etc., are shown in the tables. For example, under column 10 is 0.69, the perpendicular distance from the horizontal force h_1 to the lower-middle-third point of joint 1; 1.15 is the perpendicular distance from the horizontal force h_1 to the lower-middle-third point of joint 2; and so on.

The value of y, the moment arm of the crown thrust, is found by scaling the drawing (Fig. 154) and is recorded in Table 143. For example, 0.47 in column 18 is the perpendicular distance from the crown thrust (assumed to be applied at the upper-middle-third of the crown joint) to the lower-middle-third point of joint 1, and so on for the other values to the origin of moments of the several joints.

The next step is to find the sum of the moments of all of the vertical forces to the right of each of the origins of moments of the various joints; for joint 1, the moment of the vertical force at the right of that joint equals w_1x_1; for joint 2, the moment of the vertical forces equals $w_1x_1 +$

w_2x_2; and so on for each of the other joints. The moments of the vertical forces about each of the joints thus found are recorded in column 19.[1]

In a similar manner, find the moment of the horizontal forces about each joint and record the sum for each joint in column 20.

The total crown thrust for each joint is then found by adding the moment due to the vertical forces and the moment due to the horizontal forces and dividing by the lever arm y of the crown thrust.

$$H_1 = \frac{\Sigma wx + \Sigma hK}{y}$$

The value of the crown thrust thus obtained is recorded in column 21 of Table 143. Table 144 is similar to Table 143, with the exception that the moments are taken about the outer or upper end of the middle-third instead of the inner-third point.

A comparison of the figures in columns 21 in these tables shows that H_2 for joint 8 is less than H_1 for joint 4; therefore, the skewback cannot be at joint 8 but must be above it, for H_0 must be less than H_2 but greater than H_1. It will be noted that H_2 for joint 7 is also less than the H_1 for joint 4, but by so small an amount that we may safely consider the skewback to be at joint 7.[2] Since H_1 is greatest at joint 4, the "joint of rupture" is nearer that joint than any other and the corresponding value of H_o = 17,100 lb. If a larger number of joints had been assumed, slightly different locations of skewback and joint of rupture would probably be found.

Force Diagram.—The maximum crown thrust for the joint of rupture has already been found as 17,100 lb. To construct the force diagram, a horizontal line is drawn to scale (Fig. 154) to represent the amount of the maximum crown thrust as found for joint 4. From the left end of the horizontal line lay off w_1, the first vertical force, vertically downward, and from its extremity lay off h_1 horizontally to the right. Then the line from the right extremity of h_1 to the upper end of w_1 represents the direction and amount of the resultant external force, P_1, acting upon the first voussoir. The line R_1, drawn from the origin to the right extremity of h_1 or the lower extremity of P_1, represents the resultant pressure of the first voussoir upon the one next below it. Similarly, lay off w_2 vertically downward from the right extremity of h_1, and lay off h_2 horizontally to the right; then a line P_2 from the upper end of w_2 to the right end of h_2 represents the resultant of the external forces acting on the second voussoir, and a line R_2 from the origin to the lower extremity of P_2 represents the resultant pressure of the second voussoir on the third. The force diagram is completed by drawing lines to represent the other values of w, h, P and the corresponding reactions. The succession of

[1] NOTE: The value of x_1 with reference to joint 1 is different from x_1 with reference to joint 2. The two values of x_1 mentioned in the above paragraph are different by definition.

[2] The true skewback would be found slightly above joint 7, in a location where no joint is assumed in this discussion.

forces P_1, P_2, P_3, etc., is sometimes called the "load line," as it represents the external forces acting on the arch in direction and amount in the order of their application, starting from the crown and going toward the springing line. The radial lines from the several points on this load line to the origin are called the "rays" and represent in direction and amount the successive reactions or thrusts of one voussoir against the next.

Line of Resistance.—On the arch section through the several points of application of the horizontal and vertical forces, draw the resultant forces acting on each voussoir. These may be taken from the force diagram.

To construct the line of resistance, draw through the upper limit of the middle third of the crown joint a horizontal line to an intersection with the oblique force P_1 acting on voussoir 1; and from this point draw a line parallel to R_1 and prolong it to an intersection with the oblique force P_2 acting on voussoir 2 of the arch. In a similar manner, continue to the springing line.

On account of the method used, the line of resistance must pass through the lower-middle-third point of the joint of rupture. This offers a reliable method of checking the accuracy of the work.

The resultants R_1, R_2, etc., of the force diagram give the resultant pressure acting on each joint, and the line of resistance gives the location of the forces in the masonry at each joint. The thrust, normal to the joint, and the shear can be found by resolving the resultant pressure into its two components tangent and perpendicular to the center line of the arch ring at the point in question. This has been done graphically in the force diagram (Fig. 154), and the values may be obtained by scaling the broken lines shown in the diagram.

Having given the location and amount of the thrust on each joint, the stresses for that joint can be computed, as will be explained in the latter part of this chapter.

ANALYSIS OF ARCH BY ELASTIC THEORY

The method of analysis of an arch based on the elastic theory assumes that the arch is held in equilibrium by its resistance to combined compression and bending, that is, the arch is considered as a curved beam. This method is applicable to all hingeless arches of variable moment of inertia and to any system of loading, although the work is greatly simplified when the loads are symmetrical. As a rule, sewer arches can be considered as being symmetrically loaded.[1] The method here given is that explained by Turneaure and Maurer.

[1] For a more complete discussion of the theories and methods of analysis than is here given, the reader is referred to M. A. Howe, "Symmetrical Masonry Arches;" Ira O. Baker, "A Treatise on Masonry Construction;" Taylor, Thompson, and Smulski, "Concrete, Plain and Reinforced," Vol. II; Hool and Kinne, "Reinforced Concrete and Masonry Structures;" or Turneaure and Maurer, "Principles of Reinforced Concrete Construction."

The analysis of an arch consists of the determination of the forces acting at any joint, usually expressed as the thrust, the shear, and the bending moment at such joints. The thrust is taken to be the component of the resultant parallel to the center line of the arch ring at the given point, and the shear is the component at right angles to the latter. The thrust causes bending and direct stress, the shear causes stresses similar to those produced by the vertical shear in a simple beam.

Let H_o = thrust at the crown
V_o = shear at the crown
M_o = bending moment at the crown, assumed as positive when causing compression in the upper fibers
N, V, and M = thrust, shear, and moment at any other joint
R = resultant pressure at any joint = resultant of N and V
ds = length of a voussoir of the arch ring measured along its center line
p = number of voussoirs in one-half of the arch
I = moment of inertia of any cross-section of arch ring
w, h, P = the vertical, horizontal and resultant external loads on the arch, respectively
x, y = coordinates of any point on the center line of ring referred to the center of the crown as origin (all are positive in sign)
m = bending moment at any point in the half arch (Fig. 157) due to external loads only (all are negative in sign)

For symmetrical loads, the following equations can be derived:

$$H_o = \frac{p\Sigma my - \Sigma m\Sigma y}{(\Sigma y)^2 - p\Sigma y^2}$$

$$M_o = -\frac{\Sigma m + H_o\Sigma y}{p}$$

$$V_o = 0$$

The above calculations are for the half-arch.

The total bending moment at any joint is

$$M = m + M_o + H_o y$$

In the following analysis, based on the elastic theory and using Turneaure and Maurer's method, two cases are considered:

Case I. Arch and Side Wall, Rock Foundation.—In this case, the invert is considered as being separated from the side walls and arch, and the elastic structure to be analyzed consists only of the side wall and arch. This assumes that the base of the side wall is fixed, that is, the arch is hingeless. Such a condition would exist where the sewer is constructed in rock cut with the base of the side walls or the invert

resting on ledge rock. If the rock extended to a point above the springing line or horizontal diameter of the semicircular arch, the analysis might properly be confined to that portion of the structure above the springing line of the semicircular arch, as the ends could then be considered as fixed at that point.

Case II. Full Ring, Compressible Soil.—This case differs from the preceding in that the entire structure, invert included, is considered as an elastic monolith and consequently subject to direct stress and bending at every point. Such a condition will be reached if the sewer is constructed in compressible soil and acts as a ring. Reinforced-concrete sewers constructed in sand, gravel, or clay without special foundations should be treated under this case.

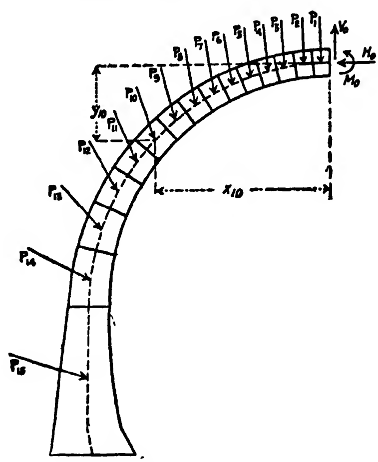


Fig. 156.—Forces and notation used in analysis.

Case I. Arch and Side Wall.—In the following discussion, the term "arch" is used to denote that portion of the sewer section from the crown to the base of the side wall or the beginning of the invert. One-half of the arch is drawn to some convenient scale, which should be sufficiently large to enable all distances to be scaled without appreciable error. The arch under consideration is shown in Fig. 156.

Division of Arch Ring to Give Constant ds/I.—The first step in the analysis is to divide the half-arch into a number of voussoirs of such length that the ratio of ds/I will be constant. The following method of determining the successive voussoirs of the arch is taken from Baker.[1] While there are a number of other methods which may be used, this is one of the simplest. Since the moments of inertia of the several divisions of the arch vary as the cube of the depth, it will be necessary to make the divisions in the side wall considerably larger than those

[1] "Treatise on Masonry Construction," Tenth Edition, 676.

near the crown, and on this account, in order to avoid excessive error, the divisions at the crown should be made comparatively small. The first step is to divide the center line of the ring into any number of equal parts, in this case 15. Measure the radial depth of the ring at each point of division; determine the length of each division either by dividers or computation, and lay off this length to scale on a horizontal line, as in Fig. 157; divide this line into the same number of equal parts as the half-arch, and at each point of division erect a vertical equal by scale to the moment of inertia at the corresponding point on the arch. Although the moment of inertia of any cross-section of the arch ring, where reinforced with steel, to be exact should be taken as the sum of the moment of inertia of the concrete section plus n times the moment of inertia of the steel section, for arches usually designed in sewerage practice it will be sufficiently accurate to consider the moment of inertia of the concrete section alone, neglecting the steel. And since the moment of inertia is proportional to the cube of the depth the latter quantity may be used for the length of the vertical line instead of the moment of inertia, as specified above. Connect the tops of these verticals by a smooth curve. It may then be assumed that any ordinate to this curve is proportional to the moment of inertia at the corresponding point on the arch ring.

To divide the arch ring into portions of such length that ds/I shall be constant, draw a line ab, at any slope and then a line, bc, to form an isosceles triangle; continue the construction by drawing other similar

TABLE 145.—DIVISION OF ARCH RING

Analysis of 15-ft. 6-in. by 15-ft. 2-in. horseshoe sewer by elastic theory

1 Joint number	2 Radial depth of ring "t"	3 t^3	4 Values of ds	1 Joint number	2 Radial depth of ring "t"	3 t^3	4 Values of ds
Crown	0.917	0.76	....	11	1.48	3.24	1.12
1	0.92	0.78	0.63	12	1.67	4.66	1.29
2	0.93	0.80	0.63	13	1.92	7.09	1.54
3	0.96	0.88	0.63	14	2.24	11.22	1.96
4	0.99	0.97	0.64	15	2.95	25.65	5.34
5	1.04	1.13	0.66	16	1.90	6.86	4.64
6	1.09	1.30	0.69	17	1.71	5.00	1.92
7	1.15	1.52	0.75	18	1.51	3.44	1.21
8	1.21	1.77	0.80	19	1.30	2.20	0.98
9	1.28	2.10	0.88	20	1.12	1.41	
10	1.35	2.46	0.98	Invert c	1.00	1.00	

Note that this table includes figure for joints 16 to 20, required in the study of Case II.

isosceles triangles as shown. This divides the arch ring into a number of parts of such length that each part, divided by the moment of inertia at its center, is constant, that is, $ds/I = 2 \tan a$, in which a is the angle between the sides of the isosceles triangle and the vertical.

In Table 145 are given the values used in the above computations for the division of the arch ring.

It is not necessary that a point of division shall fall exactly at the end of the horizontal line, but in most cases, the designer makes a few trials until a division does fall at that point. An adjustable angle or protractor will be found of considerable assistance in subdividing the arch ring in this manner.

Conditions.—The sewer section shown is assumed to be subject to an earth fill of 24 ft. above the top of the sewer. The weight of the earth filling is assumed to be 100 lb. per cubic foot, and the angle of repose is taken as 30 deg. It is further assumed that the sewer is to be constructed in rock cut with the side walls and invert resting directly on rock foundation.

Vertical Forces.—The vertical forces assumed are the weight of the masonry and the weight of the prism of earth above.[1] For purposes of analysis, the weight of the masonry can usually be omitted, where the vertical load, or the depth of earth fill, above the arch is very large. The vertical pressure of the earth above the arch is assumed to be the dead weight of the prism of earth, in width equal to the horizontal projection of the extrados of each voussoir, and in depth equal to the distance from the surface of the ground to the center of the extrados of each voussoir. In case the dead weight of the masonry is used, this can be added to the weight of the earth and the resultant pressure applied at the center of the extrados of each voussoir.

The depth, vertical intensity, horizontal projection of the voussoir, and total vertical load are tabulated in Table 146, columns 2 to 5 inclusive. Also, the vertical forces are shown graphically in their respective locations in Fig. 157.

Horizontal Forces.—If we assume the angle of repose equal to 30 deg., the intensity of the horizontal earth pressure will be one-third of the intensity of the vertical pressure at any point. The horizontal earth pressure is assumed to act on each voussoir on a width equal to the vertical projection of the extrados. The values of the horizontal intensity of the earth pressure, the vertical projection of the voussoir, and the total horizontal load are given in Table 146, columns 6, 7, and 8, and the horizontal loads are shown graphically in Fig. 157. The horizontal pressure may be assumed to act at the center of the axis for each voussoir without material error in the final results.

[1] See caution on p. 486.

TABLE 146.—COMPUTATIONS OF EXTERNAL FORCES

Analysis of 15-ft. 6-in. by 15-ft. 2-in. horseshoe sewer by elastic theory

1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
Voussoir number	Depth to center of extrados	Vertical intensity of earth pressure (pounds per square foot)	Horizontal projection of extrados of voussoir, feet	Total vertical load w, pounds	Horizontal intensity of earth pressure	Vertical projection of extrados of voussoir, feet	Total horizontal load h, pounds	Sum of vertical loads Σw	Sum of horizontal loads Σh	Coordinates of center of voussoir		y^2	Difference between successive coordinates	
										x	y		$x_2 - x_1$, etc.	$y_2 - y_1$, etc.
1	24.01	2,401	0.67	1,610	800	0 03	24	1,610	24	0.32	0.01	0.0001	0.64	0.03
2	24.06	2,406	0 66	1,587	802	0 07	56	3,197	80	0.96	0 04	0.0016	0.63	0.12
3	24.16	2,416	0.66	1,594	805	0.12	97	4,791	177	1.59	0.16	0.0256	0.60	0.13
4	24.30	2,430	0.67	1,628	810	0 18	146	6,419	323	2.19	0.29	0.0841	0.64	0.19
5	24.50	2,450	0 66	1,616	817	0 23	188	8,035	511	2.83	0.48	0.231	0.62	0.24
6	24.75	2,475	0.66	1,634	825	0.29	239	9,669	750	3.45	0.72	0.519	0.62	0.33
7	25.07	2,507	0.69	1,730	836	0.37	309	11,399	1,059	4.07	1.05	1.102	0.67	0.40
8	25.49	2,549	0.72	1,835	850	0.49	416	13,234	1,475	4.74	1.45	2.103	0.67	0.51
9	26.02	2,602	0 72	1,873	867	0.58	503	15,107	1,978	5.41	1.96	3.84	0.67	0.62
10	26.67	2,667	0.72	1,920	889	0.71	631	17,027	2,609	6.08	2.58	6.66	0.69	0.80
11	27.51	2,751	0.73	2,008	917	0.93	853	19,035	3,462	6.77	3.38	11.42	0.65	1.01
12	28.58	2,858	0.70	2,000	953	1.18	1,125	21,035	4,587	7.42	4.39	19.28	0.58	1.29
13	29.94	2,994	0.59	1,766	998	1.56	1,556	22,801	6,143	8.00	5.68	32.26	0.40	1.71
14	31.77	3,177	0.35	1,112	1,059	2.10	2,223	23,913	8,366	8.40	7.39	54.60	0.12	3.66
15	35.49	3,549	0.53	1,881	1,183	5.33	6,305	25,794	14,671	8.52	11.05	122.10	−2.25	3.73
		{3,883	0.14	546							40.63	254.23		
16	38.83	{2,680	5.64	−15,050	1,294	1.39	1,800	11,290	16,471	6.27	14.78	218.45	−3.17	0.78
17	.....	−2,680	1.92	−5,150	.... .	....	0	6,140	16,471	3.10	15.56	242.11	−1.53	0.37
18	.....	−2,680	1.20	−3,220	.....	....	0	2,920	16,471	1.57	15.93	253.76	−1.08	0.20
19	.....	−2,680	1.09	−2,920	.....	...	0	0	16,471	0.49	16.13	260.18		
											103.03 $= \Sigma y$	1,228.73 $= \Sigma y^2$		

In columns 9 and 10 are given the successive sums of the vertical and horizontal loads, respectively, at each of the voussoirs. These figures will be used later in the calculation of the moments at the different points.

The coordinates of the center points of each voussoir, x and y, referred to the center of the crown as the origin, are shown in columns 11 and 12, and the values of y^2 in column 13. Column 14 gives the values of the differences between the successive coordinates, as, for example, $(x_2 - x_1)$, $(x_3 - x_2)$, etc., and column 15 gives the differences between the y coordinates in a similar manner, as, for example, $(y_2 - y_1)$, $(y_3 - y_2)$ etc.

Bending Moments.—The bending moments (all negative) shown in column 2 of Table 147 are computed as follows:

$$
\begin{aligned}
m_1 &= 0 \\
m_2 &= w_1(x_2 - x_1) + h_1(y_2 - y_1) = (1{,}610 \times 0.64) + \\
&\qquad (24 \times 0.03) = 1{,}031 \\
m_3 &= m_2 + \Sigma w_2(x_3 - x_2) + \Sigma h_2(y_3 - y_2) = 1{,}031 + \\
&\qquad (3{,}197 \times 0.63) + (80 \times 0.12) = 3{,}054 \\
m_4 &= m_3 + \Sigma w_3(x_4 - x_3) + \Sigma h_3(y_4 - y_3) = 3{,}054 + \\
&\qquad (4{,}791 \times 0.60) + (177 \times 0.13) = 5{,}955 \\
m_5 &= m_4 + \Sigma w_4(x_5 - x_4) + \Sigma h_4(y_5 - y_4) = 5{,}955 + \\
&\qquad (6{,}419 \times 0.64) + (323 \times 0.19) = 10{,}120 \\
m_6 &= m_5 + \Sigma w_5(x_6 - x_5) + \Sigma h_5(y_6 - y_5) = 10{,}120 + \\
&\qquad (8{,}035 \times 0.62) + (511 \times 0.24) = 15{,}220 \\
m_7 &= m_6 + \Sigma w_6(x_7 - x_6) + \Sigma h_6(y_7 - y_6) = 15{,}220 + \\
&\qquad (9{,}669 \times 0.62) + (750 \times 0.33) = 21{,}470 \\
m_8 &= m_7 + \Sigma w_7(x_8 - x_7) + \Sigma h_7(y_8 - y_7) = 21{,}469 + \\
&\qquad (11{,}399 \times 0.67) + 1{,}059(\times 0.40) = 29{,}530 \\
m_9 &= m_8 + \Sigma w_8(x_9 - x_8) + \Sigma h_8(y_9 - y_8) = 29{,}530 + \\
&\qquad (13{,}234 \times 0.67) + (1{,}475 \times 0.51) = 39{,}140 \\
m_{10} &= m_9 + \Sigma w_9(x_{10} - x_9) + \Sigma h_9(y_{10} - y_9) = 39{,}140 + \\
&\qquad (15{,}107 \times 0.67) + (1{,}978 \times 0.62) = 50{,}490 \\
m_{11} &= m_{10} + \Sigma w_{10}(x_{11} - x_{10}) + \Sigma h_{10}(y_{11} - y_{10}) = 50{,}490 + \\
&\qquad (17{,}027 \times 0.69) + (2{,}609 \times 0.80) = 64{,}303 \\
m_{12} &= m_{11} + \Sigma w_{11}(x_{12} - x_{11}) + \Sigma h_{11}(y_{12} - y_{11}) = 64{,}330 + \\
&\qquad (19{,}035 \times 0.65) + (3{,}462 \times 1.01) = 80{,}200 \\
m_{13} &= m_{12} + \Sigma w_{12}(x_{13} - x_{12}) + \Sigma h_{12}(y_{13} - y_{12}) = 80{,}200 + \\
&\qquad (21{,}035 \times 0.58) + 4{,}587 \times 1.29) = 98{,}320 \\
m_{14} &= m_{13} + \Sigma w_{13}(x_{14} - x_{13}) + \Sigma h_{13}(y_{14} - y_{13}) = 98{,}320 + \\
&\qquad (22{,}801 \times 0.40) + (6{,}143 \times 1.71) = 117{,}940 \\
m_{15} &= m_{14} + \Sigma w_{14}(x_{15} - x_{14}) + \Sigma h_{14}(y_{15} - y_{14}) = 117{,}940 + \\
&\qquad (23{,}913 \times 0.12) + (8{,}366 \times 3.66) = 151{,}430 \\
m_{16} &= m_{15} + \Sigma w_{15}(x_{16} - x_{15}) + \Sigma h_{15}(y_{16} - y_{15}) = 151{,}430 + \\
&\qquad (25{,}794 \times -2.25) + (14{,}671 \times 3.73) = 148{,}120 \\
n_{17} &= m_{16} + \Sigma w_{16}(x_{17} - x_{16}) + \Sigma h_{16}(y_{17} - y_{16}) = 148{,}120 + \\
&\qquad (11{,}290 \times -3.17) + (16{,}471 \times 0.78) = 125{,}170
\end{aligned}
$$

$$m_{18} = m_{17} + \Sigma w_{17}(x_{18} - x_{17}) + \Sigma h_{17}(y_{18} - y_{17}) = 125{,}170 + (6{,}140 \times -1.53) + (16{,}471 \times 0.37) = 121{,}875$$

$$m_{19} = m_{18} + \Sigma w_{18}(x_{19} - x_{18}) + \Sigma h_{18}(y_{19} - y_{18}) = 121{,}875 + (2{,}920 \times -1.08) + (16{,}471 \times 0.20) = 122{,}019$$

Computations for m_{16} to m_{19} are for Case II.

The summations $w_1 + w_2 + w_3$ and $h_1 + h_2 + h_3$, etc., are taken from columns 9 and 10 (Table 146). The difference between the x and y coordinates, as $(x_2 - x_1)$ and $(y_2 - y_1)$, are taken from columns 14 and 15, respectively, of the same table.

Forces at Crown.—The next step in the analysis is to find the crown thrust, which can be obtained from the equation previously given. The values of Σm, Σmy, Σy, and Σy^2 are given in columns 2 and 3 of Table 147 and columns 12 and 13 of Table 146.

$$H_0 = \frac{p\Sigma my - \Sigma m\Sigma y}{(\Sigma y)^2 - p\Sigma y^2} = \frac{15(-3{,}963{,}046) - (-688{,}230 \times 40.63)}{(40.63)^2 - 15 \times 254.23}$$

$$H_0 = 14{,}562 \text{ lb.}$$

In the above equation, p = the number of divisions in the half-arch.

The bending moments at the crown can also be obtained by the equations already given, as follows:

$$M_0 = -\frac{\Sigma m + H_0\Sigma y}{p} = -\frac{-688{,}230 + 14{,}562 \times 40.63}{15}$$

$$M_0 = +6{,}438 \text{ ft. lb.}$$

The values of the crown thrust H_0 multiplied by the values of y, are computed and tabulated in column 4, Table 147.

From the data thus obtained, the total bending moment for each voussoir is computed from the formula given in a previous paragraph, as follows:

$$M = m + M_0 + H_0y$$

$$M_1 = m_1 + M_0 + H_0y_1 = 0 + 6{,}438 + 146 = +6{,}584$$

$$M_2 = m_2 + M_0 + H_0y_2 = -1{,}031 + 6{,}438 + 583 = +5{,}990$$

The results are recorded in column 5 (Table 147).

Force Diagram.—The value of the thrust and shear at any point can be obtained from the force diagram by graphical methods, as discussed on p. 491.

Line of Resistance.—The line of resistance, or diagram showing the line of pressure on the arch, is drawn by the aid of the force diagram, as was discussed on p. 491. The crown thrust acts, for a symmetrically loaded arch, in a horizontal direction, and the point of application is at a distance above center line of the ring at the crown equal to $M_0/H_0 = x_0$, the eccentric distance, if M_0 is plus, and below the axis by the same amount if M_0 is minus. These values are recorded in column 7, Table 147. For the example at hand

$$\frac{M_0}{H_0} = \frac{+6{,}438}{14{,}562} = +0.442 \text{ ft.}$$

This distance is then laid off vertically above the center line at the crown, and the resultant crown thrust is drawn through this point to its intersection with the resultant external force acting on the first voussoir. From this point of intersection draw a line parallel to R_1, taken from the force diagram, and prolong it to an intersection with the oblique force acting on voussoir 2. In a similar manner continue by taking the "rays" from the force diagram and prolong each to its intersection with the next oblique force acting on the arch. The resulting succession of lines is called the "line of resistance," or the "line of thrust" for the arch. The amount of each thrust, that is, the true thrust parallel to the center line of the ring should be scaled from the force diagram and the amounts recorded in column 6 of Table 147.

For positive moments, and, therefore, positive values of x_0, the line of thrust lies above the center line of the ring. The amount of the eccentricity is shown graphically on the diagram by the distance from the center line to the point of application of the thrust, which is the intersection of the line of resistance with a joint at the point under considera-

TABLE 147.—BENDING MOMENTS, THRUSTS, AND SHEARS. CASE I
Analysis of 15-ft. 6-in. by 15-ft. 2-in. horseshoe sewer by elastic theory

1 Voussoir number	2 Bending moment, m	3 my	4 H_0y	5 Total bending moment, foot-pounds	6 Thrust, pounds	7 Eccentric distance, feet	8 Shear, pounds
1	0	....	146	+6,584	14,550	+0.45	1,100
2	−1,031	−41	583	+5,990	14,720	+0.41	1,550
3	−3,054	−489	2,330	+5,714	14,990	+0.38	2,000
4	−5,955	−1,726	4,223	+4,706	15,400	+0.31	2,500
5	−10,120	−4,860	6,990	+3,308	15,900	+0.21	2,900
6	−15,220	−10,960	10,490	+1,708	16,530	+0.10	3,200
7	−21,470	−22,540	15,290	+258	17,330	+0.01	3,500
8	−29,530	−42,820	21,110	−1,982	18,210	−0 11	3,600
9	−39,140	−76,710	28,540	−4,162	19,330	−0.22	3,500
10	−50,490	−130,300	37,570	−6,482	20,500	−0.32	3,200
11	−64,330	−217,400	49,220	−8,672	21,860	−0.40	2,600
12	−80,200	−352,100	63,930	−9,832	23,210	−0.42	1,300
13	−98,320	−558,500	82,710	−9,172	24,300	−0.38	550
14	−117,940	−871,600	107,610	−3,892	24,500	−0 16	3,200
15	−151,430	−1,673,000	160,910	+15,918	25,800	+0.62	110
	−688,230 = Σm	−3,963,046 = Σmy					

tion. After the line of resistance has been drawn, the computed values of the eccentricity can be checked by scaling the distances from it to the center line of the arch ring. While it is not necessary to draw the "line

of resistance" in order to obtain the fiber stresses, it is usually well to do so in order to check the arithmetical work. It is to be noted that the graphical solution is not a thorough check on the work, since the line of resistance depends on the values of H_0 and M_0 for its start, and certain errors made before determining H_0 and M_0 will not show up in the graphical solution.

It should be borne in mind that the line of resistance as thus determined is only approximate. As the number of subdivisions of the ring is increased, the line of resistance approaches the true line of resistance which is a curve. The exact values of the eccentricity x_o are the distances between the center line of the ring and the curve of resistance, measured on a joint at the center points in question. For practical purposes, the line of resistance as determined is sufficiently accurate.

Case II. Full Ring.—In the preceding analysis, the invert of the ring was considered as separated from the side wall, but under Case II, the entire structure will be analyzed. The same assumptions as to vertical and horizontal forces acting on the arch and side wall are made, and in addition it is assumed that there are vertical forces acting upward on the invert equal in amount to the total downward vertical forces and uniformly distributed over the invert (see Fig. 157). The upward vertical force acting on voussoir 16 is combined with the vertical (downward) and horizontal components of the earth pressure acting on the left side of the voussoir producing the oblique resultant force as shown.

Division of Arch to Make ds/I Constant.—The chief disadvantage of this method as applied to Case II lies in the necessity of dividing the arch according to a prescribed ratio. This usually requires careful manipulation and repeated trials to subdivide the side wall and invert in order to obtain suitable divisions. It can be done as Fig. 157 shows, and in the example at hand no great difficulty was experienced. Voussoirs 15 and 16 are, however, somewhat larger than is desirable for divisions where large thrusts, bending moments, and shears occur.

The method of dividing the axis is the same as described under Case I.

Computations.—The remainder of the computations are made in the same manner as for Case I. New values of H_0, M_0, and x_0 are computed, using the summations from voussoir 1 to 19 inclusive instead of from 1 to 15 inclusive, as in Case I. For convenience, the values for points 16 to 19 have been included with the others in Table 146. The computations of the bending moments for joints 16 to 19 were given with those arising under the assumptions of Case I.

New values of H_0, M_0, and x_0 are found from the formulas as before, using the summations from voussoirs 1 to 19 inclusive, as follows:

$$H_o = \frac{p\Sigma my - \Sigma m\Sigma y}{(\Sigma y)^2 - p\Sigma y^2} = \frac{19(-12{,}009{,}546) - (-1{,}205{,}414 \times 103.03)}{(103.03)^2 - (19 \times 1{,}228.73)}$$

$$H_o = +8{,}170 \text{ lb.}$$

$$M_o = -\frac{\Sigma m + H_o\Sigma y}{p} = -\left[\frac{-1{,}205{,}414 + (8{,}170 \times 103.03)}{19}\right]$$

$$M_o = +19{,}140 \text{ ft.-lb.}$$

$$x_o = \frac{M_o}{H_o} = \frac{+19{,}140}{+8{,}170} = +2.34 \text{ ft.}$$

With the above values, a new force diagram is drawn (Fig. 157) and also a new line of resistance, following the same method as for Case I.

The computations for the bending moments, thrusts, shears, and eccentric distances are given in Table 148.

A comparison of the lines of resistance or lines of thrust for the two cases shows plainly that the bending moments are greatly increased by the addition of the invert as part of the elastic structure.

Having given the amount and point of application of the normal thrust for any joint, the resulting fiber stresses can readily be computed. The method of making these computations will be described later.

TABLE 148.—BENDING MOMENTS, THRUSTS, AND SHEARS.—CASE II Analysis of 15-ft. 6-in. by 15-ft. 2-in. Horseshoe Sewer by Elastic Theory

1 Voussoir number	2 Bending moment m, foot-pounds	3 my	4 H_oy foot-pounds	5 Total bending moment, M foot-pounds	6 Thrust, pounds	7 Eccentric distance, feet	8 Shear, pounds
1	0	0	82	19,222	8,200	+2.35	1,300
2	−1,031	−41	327	18,436	8,400	+2.20	2,250
3	−3,054	−489	1,307	17,393	8,700	+2 00	3,200
4	−5,955	−1,726	2,370	15,555	9,210	+1.69	4,200
5	−10,120	−4,860	3,920	12,940	9,870	+1.31	5,000
6	−15,220	−10,960	5,880	9,800	10,670	+0.92	5,800
7	−21,470	−22,540	8,580	6,250	11,680	+0.535	6,600
8	−29,530	−42,820	11,850	1,460	12,920	+0.11	7,200
9	−39,140	−76,710	16,010	−3,990	14,400	−0.28	7,600
10	−50,490	−130,300	21,090	−10,260	16,070	−0.64	7,800
11	−64,330	−217,400	27,610	−17,580	18,030	−0.98	7,700
12	−80,200	−352,100	35,880	−25,180	20,120	−1.25	6,950
13	−98,320	−558,500	46,400	−32,780	22,180	−1.48	5,600
14	−117,940	−871,600	60,380	−38,420	23,670	−1.62	3,200
15	−151,430	−1,673,000	90,280	−42,010	25,794	−1.63	6,500
16	−148,120	−2,189,200	120,750	− 8,230	10,600	−0.78	9,100
17	−125,170	−1,947,600	127,130	+21,100	9,400	+2.25	4,100
18	−121,875	−1,941,500	130,150	+27,415	8,660	+3.17	1,000
19	−122,019	−1,968,200	131,780	+28,901	8,200	+3.53	750
	−1,205,414 = Σm	−12,009,546 = Σmy					

ANALYSIS OF ELASTIC RING BY METHOD FOR INDETERMINATE STRUCTURES

If the sewer is constructed in compressible soil or under any conditions where it is not correct to assume that the ends of the arch are fixed, the

whole sewer section must be considered as an elastic structure subject to deformation.

The determination of the line of resistance is based on the method for computing statically indeterminate stresses in an elastic structure. This method has been ably discussed by Prof. C. W. Hudson in his "Deflections and Statically Indeterminate Stresses," on which the following discussion has been based. The authors desire to acknowledge the assistance of Arthur W. French, Professor of Civil Engineering, Worcester Polytechnic Institute, in applying this method to the analysis of sewer sections and in preparing the following notes and computations. In analyzing the ring two cases are considered.

Case I. Arch and Side Wall, Rock Foundation.—In this case, the same assumptions are made for the horizontal and vertical loads as in Case I of the previous method. It is also assumed that the thrust in the side wall goes directly into the foundation and the invert serves only as a tie or strut to space the walls, carrying none of the vertical reaction. In this case, the elastic deformation of only the semicircular arch and the side wall is considered. This method is the same in theory as the analysis for the elastic arch previously given but differs in method because, in the latter, the arch ring is divided so as to make ds/I constant, while the method described in the following pages has no restriction on the length of the voussoirs.

Case II. Full Ring, Compressible Soil.—In this case, the elastic deformation of the whole sewer ring is taken into account. There is no assumption that the ends are fixed, for the material taken symmetrically on both sides of the center of the invert is acted upon by direct stresses and bending moments (see Fig. 158). The study section, instead of acting as a cantilever beam, as in Fig. 156, acts as an elastic ring (Fig. 158). Vertical and horizontal earth pressures are assumed to act on the semicircular arch and side walls, and the upward pressure on the bottom is assumed to be uniformly distributed over the bottom and equal in total amount to the sum of the downward vertical forces. Such a distribution of the upward forces seems to be a reasonable assumption if the sewer is constructed on yielding or compressible soil, and at any rate it imposes more severe conditions than the assumption that the upward forces are distributed with greater intensity near the side walls.

There should be but little difference in the lines of resistance derived from the method given by Turneaure and Maurer and from that described in the following paragraphs. Some differences occur, however, due to the different lengths of voussoirs into which the ring is divided and the resulting difference in the position of the loads. The results by either method are probably sufficiently accurate, considering the uncertainties of loading, earth pressures, etc.

Case I. Arch and Side Wall.—In the analysis of Case I, only voussoirs 1 to 9 inclusive are used. This part of the ring is kept in equilibrium by the external loads, the reaction at the base of the side wall, and the thrust, shear, and moment at the crown. Since V_0 is zero for a symmetrically loaded arch, the values of H_0 and M_0 at the crown remain to be found.

Let H_o = thrust at the crown (Fig. 158)
V_o = shear at the crown
M_o = bending moment at the crown
m = bending moment at any joint due to the external loads on one side of the section, the ring being considered as a curved beam; negative in sign for left-hand half of arch

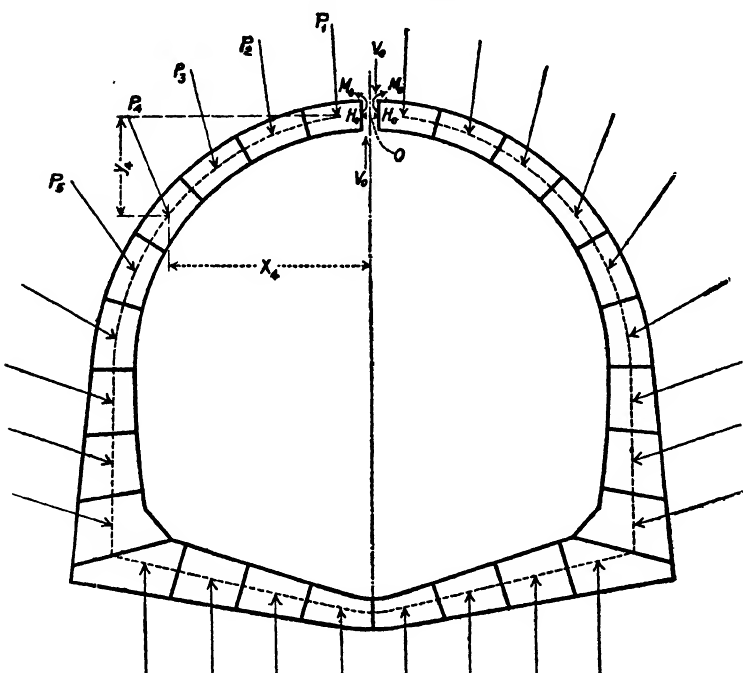


Fig. 158.—Forces and notation used in analysis.

N, V, and M = thrust, shear, and total moment at any joint
ds = length of a division of the arch ring measured along the arch axis
I = moment of inertia of any section, determined at the center
w, h, P = the vertical, horizontal, and resultant external forces, respectively, acting on the voussoir
R = the resultant pressure at any joint

x, y = coordinates of any point on the center line of the ring referred to the crown as origin (all considered as positive in sign)
m_y = moment at any joint due to 1 lb. acting vertically at O
m_x = moment at any joint due to 1 lb. acting horizontally at O
d_{1y} = vertical deflection of O due to 1 lb. acting vertically at O
d_{1x} = horizontal deflection of O due to 1 lb. acting vertically at O
d_{1a} = angular change of face at O due to 1 lb. acting vertically at O
d_{2y} = vertical deflection of O due to 1 lb. acting horizontally at O
d_{2x} = horizontal deflection of O due to 1 lb. acting horizontally at O
d_{2a} = angular change of face at O due to 1 lb. acting horizontally at O
d_{3y} = vertical deflection of O due to 1 ft.-lb. bending moment at O
d_{3x} = horizontal deflection of O due to 1 ft.-lb. bending moment at O
d_{3a} = angular change of face at O due to 1 ft.-lb. bending moment at O
Δ_y = vertical deflection of O due to external forces
Δ_x = horizontal deflection of O due to external forces
Δ_a = angular change of face at O due to external forces

Assume deflections to the left and downward as having a positive sign, and deflections in the opposite direction as negative. Assume that revolutions or angular changes of face at O in a counter-clockwise direction have a positive sign.

Equations.—From the fact that the vertical, horizontal, and angular deflection of the right and left faces of the crown joint must be identical, the three following equations can be derived:

$$\begin{aligned} +\Delta_y + V_o d_{1y} + H_o d_{2y} + M_o d_{3y} &= +\Delta_y - V_o d_{1y} + H_o d_{2y} + M_o d_{3y} \\ -\Delta_x - H_o d_{2x} - M_o d_{3x} &= +\Delta_x + H_o d_{2x} + M_o d_{3x} \\ -\Delta_a - H_o d_{2a} - M_o d_{3a} &= +\Delta_a + H_o d_{2a} + M_o d_{3a} \end{aligned}$$

From the first equation, we obtain $V_o = 0$, and on that account it has been omitted in the second and third equations.

From the second and third equations the following values can be obtained:

$$H_o = \frac{\Delta_x d_{3a} - \Delta_a d_{3x}}{d_{2a} d_{3x} - d_{2x} d_{3a}}$$

$$M_o = \frac{\Delta_x d_{2a} - \Delta_a d_{2x}}{d_{3a} d_{2x} - d_{3x} d_{2a}}$$

Considering only the deflections needed for the solution of equations for H_o and M_o, their values may be computed from the formulas

$$\left.\begin{aligned}\Delta_a &= \Sigma m\frac{ds}{EI}, \text{ used}^1 \text{ as } = \Sigma\, m\frac{ds}{t^3}\\ \Delta_x &= \Sigma m m_x\frac{ds}{EI}, \text{ used}^1 \text{ as } = \Sigma m m_x\frac{ds}{t^3}\end{aligned}\right\}$$

NOTE: Since m is negative for left half of arch, Δ_a and Δ_x are negative in sign.

$$d_{2a} = d_{3x} = \Sigma m_x\frac{ds}{EI}, \text{ used}^1 \text{ as } = \Sigma m_x\frac{ds}{t^3}$$

$$d_{3a} = \Sigma\frac{ds}{EI}, \text{ used}^1 \text{ as } = \Sigma\frac{ds}{t^3}$$

$$d_{2x} = \Sigma m_x^2\frac{ds}{EI}, \text{ used}^1 \text{ as } \Sigma m_x^2\frac{ds}{t^3}$$

where

t = thickness of masonry ring at the center of any voussoir

$$M = m + M_o + H_o y$$

In the above formulas for H_o and M_o, each expression represents the summation of the values indicated for the several divisions of the arch under consideration.

Division of Arch Ring.—The first step in the analysis is to draw the half-sewer section of suitable size to allow the scaling of various dimensions and forces with sufficient accuracy. This is shown in Fig. 159. The center line, shown dotted, is divided into a number of divisions which, for convenience, may be approximately equal, although this is not necessary. By this method, it is not necessary to subdivide the center line into such divisions that ds/I shall be constant, as in the method previously described. This has the advantage of allowing the side wall and invert to be divided into voussoirs convenient for computation, and, especially at the junction between the side wall and invert, it makes it possible to determine the bending moment with greater accuracy.

Computations.—The radial thickness of the masonry ring at the center of each voussoir is then scaled from the drawing and recorded in column 2 of Table 149. The cube of the thickness for each voussoir is recorded in column 3, and the length measured along the center line of the arch is recorded in column 4. Column 5 gives the values, for each voussoir, of ds/t^3, which is equivalent to d_{3a}.

In columns 6 and 7 of Table 149 are given the coordinates of the center point of each voussoir, the x coordinate being measured horizontally from the crown and the y coordinate being measured vertically from the center of the voussoir to the center of the crown joint.

In column 8 are given values of m_x, the moment at the center of each voussoir, due to a force of 1 lb. acting horizontally at the crown. This is equivalent to 1 lb. multiplied by the y coordinate at each center point. Column 9 gives the values of m_x^2, and column 10 the values of $m_x^2\frac{ds}{t^3}$, which will be required later for the values of d_{2x}. Column 11 gives the values of $m_x\frac{ds}{t^3}$, which will give the values of d_{2a} and its equal, d_{3x}.

[1] In the computations for H_o and M_o, after cancellation of constants appearing in both numerator and denominator.

External Forces.—Table 150 shows the computations for the external forces which are made in the same manner as the computations for the vertical and horizontal forces described under the analysis of the elastic arch (see Table 146). The sewer section shown in Fig. 159 is the same as used for the analysis of the elastic arch shown in Fig. 157. The depth of earth fill over the extrados at the crown is assumed to be 24 ft., the unit weight of earth being 100 lb. per cubic foot and the angle of repose, 30 deg.

TABLE 149.—COMPUTATIONS OF EXTERNAL FORCES AND MOMENTS

Analysis of 15-ft. 6-in. by 15-ft. 2-in. horseshoe sewer by method for indeterminate structures

1	2	3	4	5	6	7	8	9	10	11
Voussoir No.	Thickness of ring at center of voussoir, t, feet	t^3	ds, feet	$\frac{ds}{t^3}$, (d_{3a})	Coordinates of center of voussoir x, ft.	y, ft.	$m_x = y$ foot-pounds	$(m_x)^2$	$m_x^2\frac{ds}{t^3}$ (d_{2x})	$m_x\frac{ds}{t^3}$ $d_{2x} = d_{3x}$
1	0.93	0 8043	2.21	2.748	1.09	0.08	0.08	0.0064	0.018	0.220
2	0.95	0.8573	2 21	2.578	3.18	0.62	0.62	0.3844	0.991	1.598
3	1.03	1 0927	2.21	2.023	5.09	1.70	1.70	2 8900	5.847	3.439
4	1.13	1.4429	2.21	1.532	6.68	3.20	3.20	10 2410	15.688	4.902
5	1.23	1.8608	2.21	1.187	7.76	5.08	5.08	25.8030	30.630	6.030
6	1.36	2.5154	2.21	0.878	8.36	7.17	7.17	51.4090	45.140	6.294
7	1.57	3.8699	2.02	0.522	8.46	9.28	9.28	86.1180	44.950	4.844
8	1.98	7.7624	2.02	0.260	8.48	11.30	11.30	127.6900	33.200	2.938
9	2.56	16.7772	2.02	0.120	8.48	13.32	13.32	177.4220	21.290	1 599
				11.848					197.754	31.864
10	1.94	7.3014	2.18	0 299	7.43	14.59	14.59	212.8680	63.650	4.363
11	1.62	4.2515	2.18	0.513	5.32	15 07	15.07	227.1050	116.500	7.732
12	1.35	2.4604	2.18	0 886	3.20	15 55	15.55	241.8020	214.250	13.777
13	1.05	1.1576	2.18	1.883	1.07	16.03	16.03	256.9610	483.860	30.185
				15.429					1,076.014	87.921

As the method of computing the data given in Table 150, has already been explained, it will not be repeated here.

Computation of Partial Bending Moments.—In column 11, Table 150, are given the values of the differences of the coordinates, as, for example, $(x_2 - x_1)$, $(x_3 - x_2)$, etc. Column 12 gives the differences of the y coordinates. Column 13 shows the bending moments (all negative) of the external loads computed for each voussoir as follows:

$$m_1 = 0$$

$$m_2 = w_1(x_2 - x_1) + h_1(y_2 - y_1) = (5{,}475 \times 2.09) + (224 \times 0.54) = 11{,}563$$

$$m_3 = m_2 + \Sigma w_2(x_3 - x_2) + \Sigma h_2(y_3 - y_2) = 11{,}563 + (10{,}725 \times 1.91) + (912 \times 1.08) = 33{,}033$$

$$m_4 = m_3 + \Sigma w_3(x_4 - x_3) + \Sigma h_3(y_4 - y_3) = 33{,}033 + (15{,}465 \times 1.59) + (2{,}067 \times 1.50) = 60{,}723$$

TABLE 150.—COMPUTATIONS OF EXTERNAL FORCES AND MOMENTS

Analysis of 15-ft. 6-in. by 15-ft. 2-in. horseshoe sewer by method for indeterminate structures

1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
Voussoir number	Depth to center of extrados, feet	Vertical intensity of earth pressure, pounds per square foot	Horizontal projection of extrados of voussoir, feet	Total vertical load w, pounds	Horizontal intensity of earth pressure, pounds per square foot	Vertical projection of extrados of voussoir, feet	Total horizontal load h, pounds	Sum of vertical loads Σw, pounds	Sum of horizontal loads Σh, pounds	Difference between successive coordinates		Bending moment m, foot-pounds	$m\frac{ds}{t^3}$ (Δ_a)	$mmx\frac{ds}{t^3}$ (Δ_x)
										$x_2 - x_1$, etc., ft.	$y_2 - y_1$, etc , ft.			
1	24.0	2,400	2.28	5,475	800	0.28	224	5,475	224	2.09	0.54	0		
2	24.5	2,450	2.14	5,250	817	0.84	688	10,725	912	1.91	1.08	− 11,563	−30,045	−18,623
3	25.5	2,550	1.86	4,740	850	1.36	1,155	15,465	2,067	1.59	1.50	− 33,033	−66,830	−113,600
4	26.9	2,690	1.48	3,980	897	1.80	1,615	19,445	3,682	1.08	1.88	− 60,723	−93,205	−297,640
5	29.5	2,950	0.98	2,890	983	2.10	2,070	22,335	5,752	0.60	2.09	− 88,645	−105,230	−534,500
6	31.6	3,160	0.44	1,390	1,053	2.32	2,440	23,725	8,192	0.10	2.11	−114,060	−100,050	−717,960
7	33.7	3,370	0.24	810	1,123	2.10	2,360	24,535	10,552	0.02	2.02	−133,717	− 69,800	−647,700
8	35.7	3,570	0.24	857	1,190	2.20	2,620	25,392	13,172	0.00	2.02	−155,527	− 40,440	−457,000
9	37.7	3,770	0.28	1,055	1,257	2.66	3,340	26,447	16,512	−1.05	1.27	−182,137	− 21,855	−291,230
													−527,455	−3,078,253
10	....	.....	3.26	−8,700	.....	....	.....	17,747	16,512	−2.11	0.48	−175,339	− 52,430	−765,000
11	....	.....	2.16	−5,760	.....	....	.....	11,987	16,512	−2.12	0.48	−145,815	− 74,805	−1,127,500
12	....	.....	2.17	−5,790	.....	....	.....	6,197	16,512	−2.13	0.48	−128,331	−113,700	−1,768,000
13	....	.....	2.31	−6,170	.....	....	.....	0	16,512	......	....	−123,057	−231,750	−3,714,800
													−1,000,140	−10,453,553

$$m_5 = m_4 + \Sigma w_4(x_5 - x_4) + \Sigma h_4(y_5 - y_4) = 60{,}723 + (19{,}445 \times 1.08) + (3{,}682 \times 1.88) = 88{,}645$$

$$m_6 = m_5 + \Sigma w_5(x_6 - x_5) + \Sigma h_5(y_6 - y_5) = 88{,}645 + (22{,}335 \times 0.60) + (5{,}752 \times 2.09) = 114{,}060$$

$$m_7 = m_6 + \Sigma w_6(x_7 - x_6) + \Sigma h_6(y_7 - y_6) = 114{,}060 + (23{,}725 \times 0.10) + (8{,}192 \times 2.11) = 133{,}717$$

$$m_8 = m_7 + \Sigma w_7(x_8 - x_7) + \Sigma h_7(y_8 - y_7) = 133{,}717 + (24{,}535 \times 0.02) + (10{,}552 \times 2.02) = 155{,}527$$

$$m_9 = m_8 + \Sigma w_8(x_9 - x_8) + \Sigma h_8(y_9 - y_8) = 155{,}527 + (25{,}392 \times 0) + (13{,}172 \times 2.02) = 182{,}137$$

$$m_{10} = m_9 + \Sigma w_9(x_{10} - x_9) + \Sigma h_9(y_{10} - y_9) = 182{,}137 + (26{,}447 \times -1.05) + (16{,}512 \times 1.27) = 175{,}339$$

$$m_{11} = m_{10} + \Sigma w_{10}(x_{11} - x_{10}) + \Sigma h_{10}(y_{11} - y_{10}) = 175{,}339 + (17{,}747 \times -2.11) + (16{,}512 \times 0.48) = 145{,}815$$

$$m_{12} = m_{11} + \Sigma w_{11}(x_{12} - x_{11}) + \Sigma h_{11}(y_{12} - y_{11}) = 145{,}815 + (11{,}987 \times -2.12) + (16{,}512 \times 0.48) = 128{,}331$$

$$m_{13} = m_{12} + \Sigma w_{12}(x_{13} - x_{12}) + \Sigma h_{12}(y_{13} - y_{12}) = 128{,}331 + (6{,}197 \times -2.13) + (16{,}512 \times 0.48) = 123{,}057$$

$$m_{inv.} = m_{13} + \Sigma w_{13}(x_{inv.} - x_{13}) + \Sigma h_{13}(y_{inv.} - y_{13}) = 123{,}057 + (ox - 1.08) + (16{,}512 \times 0.10) = 124{,}708$$

From the values of the moments given in column 13, Table 150, and the data in columns 5 and 11, Table 149, the values of Δ_a and Δ_x shown in columns 14 and 15, of Table 150, can be computed.

Crown Thrust.—From the data at hand it is now possible to compute the value of H_o, the crown thrust, from the formula previously given,

$$H_o = \frac{\Delta_x d_{3a} - \Delta_a d_{3x}}{d_{2a}d_{3x} - d_{2x}d_{3a}} = \frac{-(3{,}078{,}253 \times 11.848) + (527{,}455 \times 31.864)}{(31.864 \times 31.864) - (197.754 \times 11.848)}$$

$$H_o = +14{,}810$$

Moment at the Crown.—The moment at the crown, M_o, can also be computed from the formula already given, as follows:

$$M_o = \frac{\Delta_x d_{2a} - \Delta_a d_{2x}}{d_{3a}d_{2x} - d_{3x}d_{2a}} = \frac{-(3{,}078{,}253 \times 31.864) + (527{,}455 \times 197.754)}{(11.848 \times 197.754) - (31.864 \times 31.864)}$$

$$M_o = +4{,}680$$

Eccentricity at the Crown.—From the values just computed, the eccentricity at the crown, x_o, can be obtained as follows:

$$x_o = \frac{M_o}{H_o} = \frac{+4{,}680}{14{,}810} = 0.316 \text{ ft.}$$

If M_o is positive, the value of x_o will also be positive and the distance x_o from the center line to the point of application of the crown thrust should be measured vertically upward. If, on the other hand, the value of M_o is negative, the corresponding value of the eccentric distance x_o will be negative, and the eccentric distance should, in that case, be measured vertically downward.

The total bending moments M at each center point can now be computed from the formula

$$M = m + M_o + H_o y$$

Column 3, Table 151, gives the values thus obtained. The fact should be borne in mind that for the left half of the structure (the half considered in this analysis) the values of the bending moment m all have a negative sign.

TABLE 151.—BENDING MOMENTS, THRUSTS, AND SHEARS—CASE I

Analysis of 15-ft. 6-in. by 15-ft. 2-in. horseshoe sewer by method for indeterminate structures

1 Voussoir number	2 $H_o y$, foot-pounds	3 Total bending moment, foot-pounds	4 Thrust, N, pounds	5 Eccentric distance, x_o, feet	6 Shear, V, pounds
Crown	....	+ 4,680	14,840	+0.32	
1	1,180	+ 5,860	15,120	+0.39	3,500
2	9,190	+ 2,307	16,830	+0.14	4,700
3	25,190	− 3,163	19,400	−0.16	4,650
4	47,400	− 8,643	22,000	−0.39	3,300
5	75,300	− 8,665	24,000	−0.36	750
6	106,200	− 3,180	24,400	−0.13	2,800
7	137,400	+ 8,363	24,535	+0.34	4,200
8	167,400	+16,553	25,390	+0.65	1,500
9	197,300	+19,843	26,450	+0.75	1,700

Force Diagram.—From the data at hand, the force diagram can be constructed in the same manner as described under the analysis of the elastic arch. The stress at the crown, $H_o = +14{,}810$, is laid off on a horizontal line, as shown in Fig. 159, and the load line of the external forces constructed, from which the resultants, thrusts, and shears can be obtained. The thrusts and shears are entered in columns 4 and 6 of Table 151.

Line of Resistance.—The line of resistance can now be constructed in the same manner as described under the analysis by the elastic theory. The crown thrust is located at a distance from the center line equal to the eccentric distance already found, or 0.316 ft. This distance is laid off vertically upward from the arch center line at the crown and the crown thrust is extended to its intersection with the first oblique external force acting on voussoir 1 of the arch. The remainder of the line of resistance can be constructed in accordance with the analysis by the elastic theory already described.

As previously stated, it is not necessary to draw the line of resistance in order to obtain the stresses at the various points but it is usually advisable to do so in order to obtain the advantage of checking the algebraic work by scaling the eccentric distances from the diagram, for comparison with those computed and recorded in column 5 of Table 151. If these distances, or any one or more of them, do not check with reasonable accuracy, the computation should be inspected for possible errors.

The check is not absolute, however, as explained under the previous method.

Case II. Full Ring.—The half-section shown in Fig. 159 may be considered as a curved beam acted upon by the known external loads and the unknown values of H_o, V_o, M_o. If these three unknown forces are determined, the resultant force acting at any joint may be found, either analytically or graphically.

$$H_o = \frac{\Delta_x d_{3a} - \Delta_a d_{3x}}{d_{2a} d_{3x} - d_{2x} d_{3a}} = \frac{-(10{,}453{,}553 \times 15.429) + (1{,}000{,}140 \times 87.921)}{(87.921 \times 87.921) - (1{,}076.014 \times 15.429)}$$

$$H_o = +8{,}260$$

$$M_o = \frac{\Delta_x d_{2a} - \Delta_a d_{2x}}{d_{3a} d_{2x} - d_{3x} d_{2a}} = \frac{-(10{,}453{,}553 \times 87.921) + (1{,}000{,}140 \times 1{,}076.014)}{(15.429 \times 1{,}076.014) - (87.921 \times 87.921)}$$

$$M_o = +17{,}700$$

The eccentric distance is obtained as before:

$$x_o = \frac{M_o}{H_o} = \frac{17{,}700}{8{,}260} = 2.14.$$

TABLE 152.—BENDING MOMENTS, THRUSTS AND SHEARS—CASE II

Analysis of 15-ft. 6-in. by 15-ft. 2-in. horseshoe sewer by method for indeterminate structures

1 Voussoir number	2 $H_o y$ foot-pounds	3 Total bending moment, M, foot-pounds	4 Thrust, N, pounds	5 Eccentric distance, x_o, feet	6 Shear, V, pounds
Crown	....	17,700	8,260	+2.14	0
1	660	18,360	8,670	+2.11	4,400
2	5,120	11,257	10,750	+1.04	7,200
3	14,050	− 1,283	14,200	−0.10	8,600
4	26,410	−16,613	18,000	−0.93	8,500
5	41,990	−28,955	21,400	−1.36	6,750
6	59,200	−37,160	23,400	−1.59	3,700
7	76,650	−39,367	24,600	−1.60	2,300
8	93,400	−44,427	25,400	−1.75	5,000
9	110,000	−54,437	26,450	−2.06	8,250
10	120,500	−37,139	12,100	−3.08	15,500
11	124,500	− 3,615	10,800	−0.34	9,850
12	128,500	+17,869	9,480	+1.87	4,200
13	132,400	+27,043	8,100	+3.33	1,800
Invert	133.250	+26,242	8,250	+3.18	0

A force diagram can now be constructed, using the same load line as before (Fig. 159). The value of H_o, the horizontal thrust at the crown, will be different, and on that account the length and direction of the rays will be different.

From this force diagram, another line of resistance can be laid out on the masonry section, as shown in Fig. 159, the dash line being the line of resistance for Case I, while the dash-dot line is the line of resistance for Case II.

It is of interest to compare the two lines of resistance, as showing the difference in the internal stresses set up in the masonry on account of the change in the assumption of the action of the masonry invert. The stresses in the masonry, where the sewer is constructed as a monolith from invert to crown and the invert rests on compressible soil, are much higher in crown, invert (especially in the latter), and side walls than in the case where the side walls and invert rest on ledge foundation.

Referring to Fig. 159, it will be noted that between voussoirs 9 and 10 the line of resistance doubles back on itself. Obviously, the true (curved) line of resistance cannot have such a shape, and the line developed by the analysis is affected by the locations chosen for joints and the corresponding size of voussoirs. The particular (irregular) form is of no special importance but is liable to be confusing unless special care is taken in scaling the eccentric distances from the center line of the ring to the correct lines of resistance. If the line of resistance from one external force to the next is followed in logical order, there should be no trouble. A different arrangement of the voussoirs or the choice of a greater number of them would probably result in eliminating this peculiarity without changing the location of the lines in the remaining voussoirs.

It is interesting to note that the line of resistance lies outside the masonry for almost its entire distance, and at the invert it is considerably below it.

COMPARISON OF RESULTS OF ANALYSES BY THREE DIFFERENT METHODS

In order to make a comparison of the three methods of analyzing arch stresses, some computations have been made, the results of which are shown in the following table. These results compare favorably considering the somewhat different conditions involved.

In the following table, the results by the static method were computed considering the abutment at the end of voussoir 7 (Fig. 154). The other two methods were computed with the assumed abutment at the end of voussoir 13 (Fig. 157). Attention should be called to the fact that in the last two methods the external loads were assumed to act at the center of the voussoirs, while in the static method the loads act at the center of the horizontal and vertical projections of the outside surface of the voussoirs. This position of the loads would tend to give a larger moment about the abutment for the elastic and indeterminate methods and correspondingly larger crown thrust.

It is interesting to note that if the arch is figured by the method of indeterminate structures through voussoir 5 of Fig. 159, which corresponds very closely with the end of voussoir 13 of Fig. 157, the value

TABLE 153.—COMPARISON OF THE THRUST AND MOMENT AT THE CROWN OF THE HORSESHOE SECTION BY THE THREE METHODS

	Static method	Elastic theory	Method for indeterminate structures
H_o, pounds	17,100	17,700	17,570
M_o, foot-pounds	2,615	2,030	2,358

of H_o = 18,040 lb. and M_o = 367 ft.-lb. This serves to illustrate the fact that if the arch is divided into too few voussoirs the results will not be accurate.

Table 154 shows how the elastic theory and the method of indeterminate structures compare for Cases I and II. The comparison would appear to be favorable when we consider that for the method of indeterminate structures the full ring is divided into only 13 voussoirs against 19 for the elastic method, with 9 and 15 divisions, respectively, for Case I.

TABLE 154.—COMPARISON OF RESULTS BY THE ELASTIC AND INDETERMINATE METHODS FOR CASES I AND II

Case		Elastic theory	Method for indeterminate structures
I	H_o, pounds	14,560	14,810
I	M_o, foot-pounds	6,438	4,680
II	H_o, pounds	8,170	8,260
II	M_o, foot-pounds	19,140	17,700

It would have been more accurate in the case of the indeterminate method to have made 18 or 19 divisions, but for the purpose of saving space and computations it has been figured with only 13 voussoirs. There is no restriction on the length of voussoirs for this method, but the designer should bear in mind the fact that with too few divisions of the arch ring, the results obtained are less accurate.

CRACKS IN LARGE SEWER SECTION

Some years ago, the attention of the authors was called to the action of a large horseshoe sewer section which had cracked in the arch (Fig.

160). This section, although slightly smaller than the section analyzed in the foregoing discussion, was of practically the same type and was constructed as a reinforced-concrete monolithic structure on compressible soil. It will be noted that the arch cracked, as might be expected from a study of the line of resistance for Case II, where the stresses in the steel were excessive and the stresses in the concrete exceeded the ultimate strength. The locations of the cracks shown were obtained by measurement. It is probable that cracks also occurred in the invert, although no definite information was obtained on account of the flow of sewage. While this structure did not fail nor was it distorted to any noticeable degree, yet the small cracks shown in the section could easily be detected and showed clearly that the steel had stretched sufficiently to allow the concrete to crack.

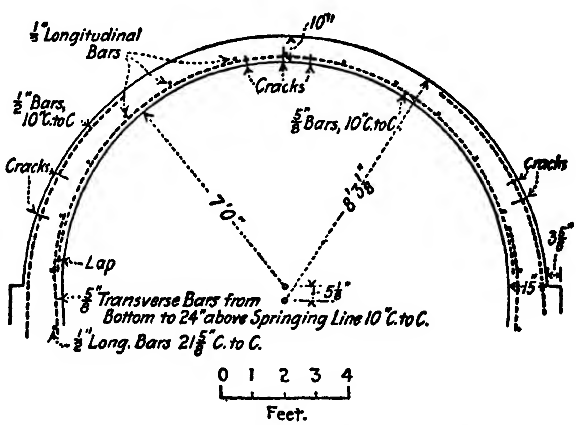


FIG. 160.—Cracks in sewer arch caused by excessive loading.

Obviously, cracks large enough to permit the passage of water are objectionable, both because they may allow water to pass into or out from the conduit, and because they would permit water to come in contact with the steel, which would rust and lose its value as reinforcement. Cracks of considerable size may also be evidence of partial failure of the structure, particularly if they indicate that the bond between concrete and steel has been destroyed.

Very fine cracks which cannot admit water to the steel are usually unobjectionable, although they may sometimes detract from the appearance of the finished works. Usually, such cracks can be detected only by very close examination, in which case they are unlikely to affect the appearance.

ANALYSIS OF 15½-FT. SEMIELLIPTICAL SEWER SECTION BY METHOD FOR INDETERMINATE STRUCTURES

As an example of the analysis of a different type of structure from that previously shown, the following analysis of a 15-ft. 6-in. semielliptical type of sewer section will be of interest. The computations are given in Tables 155, 156, and 157, and the arch section, the force diagram, and the line of resistance are shown in Fig. 161. As the method of analysis is exactly the same as that described for the horseshoe section, no detailed explanation is necessary.

Conditions.—The sewer shown in Fig. 161 is of the general type shown in Fig. 118. It is assumed that the depth of earth fill over the crown of the sewer is 24 ft., that the weight of the earth filling is 100 lb. per cubic foot, and that the angle of repose of the earth filling is 30 deg. It is further assumed that the sewer is to be built in compressible soil without the use of piles or a timber platform. If the upward forces on the invert are assumed normal to the bottom of the masonry, the condition will be less severe than as shown.

TABLE 155.—COMPUTATIONS OF EXTERNAL FORCES AND MOMENTS

Analysis of 15-ft. 6-in. semielliptical sewers by method for indeterminate structures

1	2	3	4	5	6	7	8	9	10	11
Voussoir number	Thickness of ring at center of voussoir, t, feet	t^3, feet	ds, feet	$\frac{ds}{t^3}$ (d_{3a}) feet	Coordinate of center of voussoir		$m_x = y$ by 1 lb.	m_x^2	$m_x^2\frac{ds}{t^3}$ (d_{2x})	$m_x\frac{ds}{t^3}$ ($d_{2a} = d_{3x}$)
					x, feet	y, feet				
1	1.30	2.197	2.41	1.097	1.21	0.13	0.13	0.017	0.019	0.143
2	1.30	2.197	2.41	1.097	3.40	1.10	1.10	1.210	1.327	1.208
3	1.30	2.197	2.40	1.092	5.06	2.85	2.85	8.123	8 869	3.112
4	1.39	2.686	2.39	0.890	6.37	4.84	4.84	23.426	20.850	4.308
5	1.51	3.443	2.39	0.694	7.40	6.98	6.98	48.720	33.811	4.844
6	1.68	4.742	2.39	0.504	8.17	9.25	9.25	85.563	43.126	4.662
7	1.86	6.435	2.39	0.371	8.63	11.60	11.60	134.560	49.925	4.304
8	2.00	8.000	2.39	0.299	8.77	13.96	13.96	194.882	58.270	4.175
9	1.94	7.301	1.81	0.248	7.89	15.52	15.52	240.870	59.730	3.850
10	1.94	7.301	1.80	0.247	6.19	16.15	16.15	260.823	64.400	3.990
11	1.94	7.301	1.80	0.247	4.44	16.61	16.61	275.892	68.120	4.102
12	1.94	7.301	1.80	0.247	2.69	16.92	16.92	286.286	70.710	4.179
13	1.94	7.301	1.80	0.247	0.90	17.07	17.07	291.385	71.972	4.216
				7.280					551.129	47.093

TABLE 156.—COMPUTATIONS OF EXTERNAL FORCES AND MOMENTS

Analysis of 15-ft. 6-in. semi-elliptical sewer by method for indeterminate structures

1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
Voussoir number	Depth to center of extrados, feet	Vertical intensity of earth pressure, pounds per square foot	Horizontal projection of extrados of voussoir, feet	Total vertical load w, pounds	Horizontal intensity of earth pressure, pounds per square foot	Vertical projection of extrados of voussoir, feet	Total horizontal load h, pounds	Sum of vertical loads Σw, pounds	Sum of horizontal loads Σh, pounds	Difference between successive coordinates		Bending moment m, foot-pounds	$m\frac{ds}{t^3}$ Δ_c	$mm_s\frac{ds}{t^3}$ Δ_s
										$x_2 - x_1$, etc , feet	$y_2 - y_1$, etc., feet			
1	24.15	2,415	2.60	6,279	805	0.55	443	6,279	443	2 19	0.97			
2	25.25	2 525	2.15	5,429	842	1.55	1,305	11,708	1,748	1.66	1.75	− 14,182	−15,558	− 17,132
3	27.10	2,710	1.52	4,119	903	1.95	1,761	15,827	3,509	1.31	1.99	− 36,676	−40,051	−114,140
4	29.12	2,912	1.30	3,786	971	2.12	2,058	19,613	5,567	1.03	2 14	− 64,392	−57,310	−277,400
5	31.32	3,132	1.00	3,132	1,044	2.28	2,380	22,745	7,947	0.77	2.27	− 96,506	−66,970	−467,475
6	33.68	3,368	0.72	2,425	1,123	2.40	2,695	25,170	10,642	0.46	2.35	−132,060	−66,560	−615,660
7	36.20	3,620	0.40	1,448	1,207	2 48	2,993	26,618	13,635	0.14	2.36	−168,647	−62,565	−725,870
8	38.83	3,883	0.10	388	1,294	3.00	3,882	27,006	17,517	−0.88	1.56	−204,553	−61,165	−853,950
9	.....	−2,762	2.40	−6,629	.... .	....	.....	20,377	17,517	−1.70	0.63	−208,080	−51,603	−800,880
10	.....	−2,762	1.80	−4,972	.....	....	.....	15,407	17,517	−1.75	0.46	−184,516	−45,575	−736,040
11	.....	−2,762	1.83	−5,054	.....	....	.....	10,354	17,517	−1.75	0.31	−165,574	−40,898	−679,315
12	.....	−2,762	1.85	−5,110	.....	....	.....	5,246	17,517	−1.79	0.15	−152,884	−37,762	−638,440
13	.....	−2,762	1.90	−5,246	.....	....	.....	0	17,517	......	....	−146,132	−36,092	−616,000
													−582,109 $= \Delta_c$	−6,542,302 $= \Delta_s$

TABLE 157. — BENDING MOMENTS, THRUSTS, AND SHEARS

Analysis of 15-ft. 6-in. semielliptical sewer by method for indeterminate structures

1 Voussoir number	2 H_oy, foot pounds	3 Total bending moment, M, foot pounds	4 Thrust, N, pounds	5 Eccentric distance, x_o, feet	6 Shear, V, pounds
Crown	....	7,100	11,255	0.631	0
1	1,463	8,563	11,870	0.722	3,950
2	12,375	5,293	14,600	0.363	3,900
3	32,070	2,494	17,260	0.145	3,630
4	54,460	− 2,832	19,800	−0.143	5,250
5	78,530	−10,876	22,150	−0.491	6,140
6	104,100	−20,860	24,320	−0.858	6,580
7	130,550	−30,997	26,100	−1.188	5,800
8	157,150	−40,303	26,900	−1.498	6,350
9	174,600	−26,380	13,420	−1.966	16,350
10	181,750	4,334	10,450	0.415	12,750
11	186,900	28,426	8,350	3.405	8,550
12	190,500	44,716	6,900	6.480	4,300
13	192,100	53,068	6,262	8.475	
C.L. inv.	193,000	53,267[1]	6,262	8.500	0

[1] Obtained in same manner as other Total Bending Moments, $M = m + M_0 + H_0y_{inv}$. or by scaling x_0 for thrust at center of invert then $m_{inv.} = 6{,}262 \times 8.50 = 53{,}200$. The subscript "inv." indicates measurements dealing with the center line of the invert.

BENDING MOMENTS

All negative

$m_1 = 0$

$m_2 = (6{,}279 \times 2.19) + (443 \times 0.97) = 14{,}182$

$m_3 = 14{,}182 + (11{,}708 \times 1.66) + (1{,}748 \times 1.75) = 36{,}676$

$m_4 = 36{,}676 + (15{,}827 \times 1.31) + (3{,}509 \times 1.99) = 64{,}392$

$m_5 = 64{,}392 + (19{,}613 \times 1.03) + (5{,}567 \times 2.14) = 96{,}506$

$m_6 = 96{,}506 + (22{,}745 \times 0.77) + (7{,}947 \times 2.27) = 132{,}060$

$m_7 = 132{,}060 + (25{,}170 \times 0.46) + (10{,}642 \times 2.35) = 168{,}647$

$m_8 = 168{,}647 + (26{,}618 \times 0.14) + (13{,}635 \times 2.36) = 204{,}553$

$m_9 = 204{,}553 + (27{,}006 \times -0.88) + (17{,}517 \times 1.56) = 208{,}080$

$m_{10} = 208{,}080 + (20{,}377 \times -1.70) + (17{,}517 \times 0.63) = 184{,}516$

$m_{11} = 184{,}516 + (15{,}407 \times -1.75) + (17{,}517 \times 0.46) = 165{,}574$

$m_{12} = 165{,}574 + (10{,}354 \times -1.75) + (17{,}517 \times 0.31) = 152{,}884$

$m_{13} = 152{,}884 + (5{,}246 \times -1.79) + (17{,}517 \times 0.15) = 146{,}132$

$m_{inv.} = 146{,}132 + (0 \times -0.90) + (17{,}517 \times 0.04) = 146{,}833$

$$H_0 = \frac{-(6{,}542{,}302 \times 7.280) + (582{,}109 \times 47.093)}{(47.093 \times 47.093) - (551.129 \times 7.280)}$$

$$H_0 = +11{,}255$$

$$M_0 = \frac{-(6{,}542{,}302 \times 47.093) + (582{,}109 \times 551.129)}{(7.280 \times 551.129) - (47.093 \times 47.093)}$$

$$M_0 = +7{,}100$$

$$x_0 = \frac{M_0}{H_0} = \frac{7{,}100}{11{,}255} = +0.631$$

DRESSER FORMULAS

The work involved in the analysis of stresses in a sewer section is very considerable. In the ordinary application of the elastic theory or of the

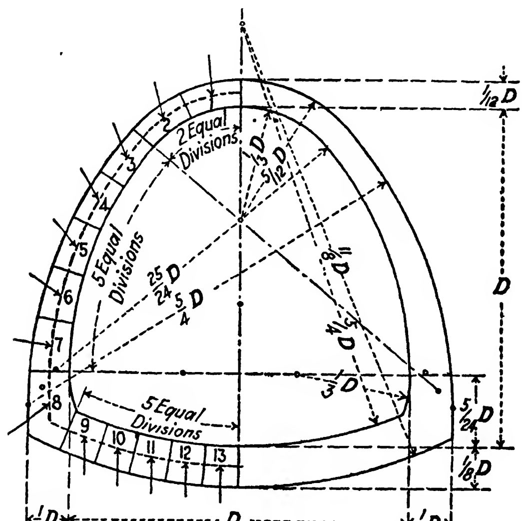


Fig. 162.—Division of authors' semielliptical section into voussoirs for application of Dresser formulas.

method for indeterminate structures, all the work must be repeated for each analysis.

If all the dimensions of a sewer section bear definite relations to the diameter, formulas can be derived for moment, thrust, and shear at any point in the section (joint), corresponding to certain conditions of loading. The solution of these equations for the loads corresponding to various depths of trench, and for various sizes of sewer, involves very much less work than would be required for the usual analyses. The derivation of the formulas is laborious, however, and unless they were to be applied repeatedly to a standard form of section, the work would not be justified.

The fact that such formulas may be derived and utilized has been shown by Herman G. Dresser, who has developed formulas for the authors' semielliptical section (Fig. 118), utilizing the method for indeterminate structures. They are applicable when the load to be carried is the equivalent of that contained between vertical planes touching the outside of the masonry (see pages 484 and 486) and when the horizontal component of the earth pressure is one-third of the vertical.

For the derivation of the formulas, the half-ring was divided into 13 voussoirs, as shown in Fig. 162. The formulas, listed in Table 158, give the moment, thrust, and shear at the crown and at the center of each of the voussoirs; p represents the ratio of depth of fill above the crown (including other loads converted to equivalent depth of fill) to diameter of sewer, w the unit weight of backfill (pounds per cubic foot), and D the diameter of the sewer in feet. The moment is positive when it causes compression in the outside fibers and negative when it causes compression in the inside fibers. Formulas are given for Case I (arch and side wall only, with rigid support under side wall) and

TABLE 158.—DRESSER'S FORMULAS FOR STRESSES IN METCALF AND EDDY SEMIELLIPTICAL SEWER SECTION

Case I.—Arch and side wall

Point	Moment, ft.-lb.	Thrust, lb.	Shear, lb.
Crown	$(-.0014p - .0064)wD^3$	$(.3537p + .0581)wD^2$	Zero
1	$(.0018p - .0059)wD^3$	$(.3701p + .0568)wD^2$	$(.0979p - .0123)wD^2$
2	$(-.0009p - .0020)wD^3$	$(.4341p + .0511)wD^2$	$(.0669p - .0239)wD^2$
3	$(.0010p + .0031)wD^3$	$(.4880p + .0534)wD^2$	$(.0381p - .0178)wD^2$
4	$(-.0001p + .0063)wD^3$	$(.5361p + .0667)wD^2$	$(.0515p + .0021)wD^2$
5	$(-.0027p + .0067)wD^3$	$(.5777p + .0854)wD^2$	$(.0483p + .0264)wD^2$
6	$(-.0038p + .0036)wD^3$	$(.6084p + .1069)wD^2$	$(.0299p + .0527)wD^2$
7	$(-.0018p - .0033)wD^3$	$(.6231p + .1256)wD^2$	$(.0057p + .0827)wD^2$
8	$(.0091p - .0155)wD^3$	$(.6166p + .1462)wD^2$	$(.0647p + .1130)wD^2$

Case II.—Full ring

Point	Moment, ft.-lb.	Thrust, lb.	Shear, lb.
Crown	$(.0171p - .0076)wD^3$	$(.2634p + .0639)wD^2$	Zero
1	$(.0195p - .0070)wD^3$	$(.2818p + .0625)wD^2$	$(.1170p - .0135)wD^2$
2	$(.0107p - .0027)wD^3$	$(.3622p + .0557)wD^2$	$(.1215p - .0274)wD^2$
3	$(.0021p + .0031)wD^3$	$(.4338p + .0569)wD^2$	$(.1103p - .0225)wD^2$
4	$(-.0105p + .0071)wD^3$	$(.4920p + .0695)wD^2$	$(.1303p - .0030)wD^2$
5	$(-.0256p + .0082)wD^3$	$(.5445p + .0876)wD^2$	$(.1323p + .0210)wD^2$
6	$(-.0399p + .0060)wD^3$	$(.5869p + .1083)wD^2$	$(.1175p + .0470)wD^2$
7	$(-.0515p - .0001)wD^3$	$(.6133p + .1262)wD^2$	$(.0954p + .0769)wD^2$
8	$(-.0566p - .0112)wD^3$	$(.5294p + .1249)wD^2$	$(.0351p + .1095)wD^2$
9	$(-.0129p - .0110)wD^3$	$(.2304p + .1458)wD^2$	$(.3636p + .0430)wD^2$
10	$(.0255p - .0059)wD^3$	$(.1729p + .1359)wD^2$	$(.2815p + .0330)wD^2$
11	$(.0555p - .0019)wD^3$	$(.1297p + .1284)wD^2$	$(.1912p + .0213)wD^2$
12	$(.0755p + .0007)wD^3$	$(.1016p + .1236)wD^2$	$(.0956p + .0085)wD^2$
13	$(.0858p + .0021)wD^3$	$(.0895p + .1217)wD^2$	Zero

Case II (entire ring in compressible earth). The formulas for Case I should be used only when the structure is to be built on solid rock.

If the maximum compressive stress is excessive, it will be necessary to increase the thickness of the ring. If the required increase is not great, the design may still be based upon the moments, thrusts, and shears given by the formulas, on the assumption that the change in thickness of the cross-section will not materially affect their applicability. If considerable increase in thickness is needed, however, it will be necessary to analyze the stresses by one of the methods previously explained, the Dresser formulas not being applicable.[1]

In the case of a wide trench, the loading to be sustained will be materially greater than that assumed in the Dresser formulas, and they should not be employed.

COMPUTATION OF STRESSES IN ARCH

In the previous discussions, the thrust, shear, and bending moment have been computed for the various divisions of the arch ring (voussoirs). The next step in the design of the sewer arch is to determine the maximum stresses in the masonry and steel in order to make sure that they do not exceed the safe working stresses and, further, to determine that the arch has been designed as economically as possible.

In order to simplify the discussion, plain concrete or masonry will be considered separately from concrete reinforced with steel.[2]

Let R = resultant of all forces acting on any cross-section of arch ring
f_c = maximum unit compression in masonry
f'_c = minimum unit compression in masonry
N = thrust, the component of R normal to the joint
V = shear, the component of R parallel to the joint
b = breadth of rectangular cross-section, taken as 12 in.
t = thickness or depth of rectangular cross-section, in inches[3]
x_o = eccentricity, that is, the distance from the center line of ring to the point of application of the thrust, which is the intersection of the line of resistance with the plane of the cross-section
M = bending moment on the cross-section = Nx_o
f'_s = maximum unit compression in the steel

[1] It would be possible to compute another set of formulas for a thick section, similar to the Dresser formulas for the standard section, using the second set when it became necessary to employ a section too thick for the application of the Dresser formulas; but the likelihood of making use of them seems too small to warrant the labor required.

[2] Discussions similar to the following may be found in HOOL and JOHNSON, "Concrete Engineers' Handbook;" HOOL and KINNE, "Structural Members and Connections;" and TAYLOR, THOMPSON, and SMULSKI, "Concrete, Plain and Reinforced," Vol. I.

[3] It should be noted that in previous pages dealing with external loads, dimensions are in feet and moments in foot-pounds. In considering stresses, the inch is the customary unit.

f_s = maximum unit tension or minimum unit compression in the steel
p_o = ratio of steel area at both faces to total area of cross section,
= for rectangular sections, ratio of steel area to bt
n = E_s/E_c = ratio of moduli of elasticity of steel and concrete
k = ratio of depth of neutral axis to depth t
kt = distance from outside compressive surface to neutral axis
d' = depth of steel in compression, from compressive surface
d = depth of steel in tension, from compressive surface
r = distance from center of gravity of symmetrical section to steel
A_s = area of steel near face least highly stressed in compression
A'_s = area of steel near face most highly stressed in compression

Stresses in Plain Concrete or Masonry Arch Section.—Sewer arches constructed of plain concrete or masonry should be so designed that the

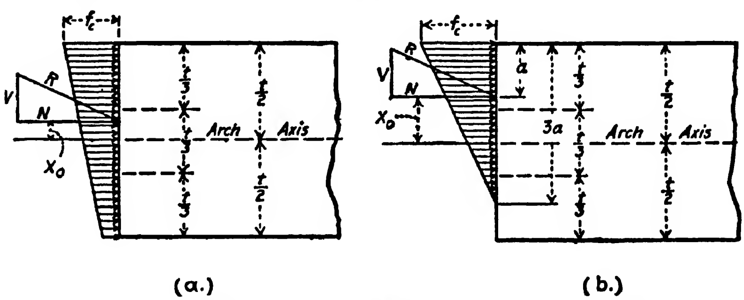


FIG. 163.—Stresses caused by forces acting on plain concrete section.

line of resistance will not lie outside the middle third of the masonry at any point. It is assumed in the design that plain concrete, brick, or stone masonry cannot resist tensile stresses and that on that account the line of resistance should lie within the middle third, so that there will be nothing but compressive stresses developed.

The general formulas for the compressive stresses, both maximum and minimum, in any section of the arch ring, are as follows (see a, Fig. 163):

$$\text{Maximum} = f_c = \frac{N}{bt}\left(1 + \frac{6x_o}{t}\right)$$

$$\text{Minimum} = f'_c = \frac{N}{bt}\left(1 - \frac{6x_o}{t}\right)$$

These general formulas apply to rectangular sections and will hold as long as the safe tensile strength of the concrete or masonry is not exceeded. As previously stated, however, no tension should be allowed to exist in the masonry. In the examination of arches already constructed, it sometimes happens that the line of resistance is found to

be outside of the middle third, and since it is assumed that the material is unable to carry tension, the preceding formula is not applicable for computing the stresses on the section. In this case, the stress is distributed as compression over a depth less than the entire depth of the section, and cracks may be expected on the tension side (see b, Fig. 163). The maximum compression in this case equals $f_c = 2N/3ba$ where a = distance from point of application of thrust to most compressed surface.

Stresses in Concrete Section Reinforced at Both Faces Symmetrically. In reinforced-concrete sections, the area of steel in compression can be replaced in the design by an equal area of concrete by multiplying the steel area by n, the ratio of modulus of elasticity of steel to the modulus of concrete. The moments of inertia may also be compared in a similar manner, and the section treated in the design as if it were entirely composed of concrete. In the design of a reinforced-concrete section, it is assumed that the concrete is not allowed to carry tension but that all of the tensile stresses must be carried by the steel reinforcement.

Compression over Entire Section.—The following equation expresses the approximate value of the maximum unit compression in the concrete under conditions where no tensile stresses exist in the section (see Fig. 164).

$$f_c = \frac{N}{bt}\left[\frac{1}{1 + np_o} + \frac{x_o}{t}\,\frac{6}{1 + 12np_o\left(\frac{r}{t}\right)^2}\right]$$

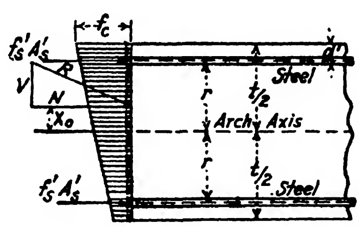


FIG. 164.—Stresses caused by a force producing compression upon the whole reinforced section.

This condition does not necessarily mean that the line of pressure lies at or within the limits of the middle third of the section, for in a reinforced-concrete section the value of the eccentricity x_o, at which there is neither compression nor tension at the surface opposite to that on which the thrust acts, is usually somewhat greater than $t/6$. For greater values of the eccentricity, and assuming that the concrete is unable to carry any tension, the above formula is not applicable. Limiting values of the ratio x_o/t are given in Table 159 for $n = 15$ and various values of p_o.

If the value of x_o/t is less than that shown in the table, the above formula will apply and no tension exists in the section.

TABLE 159.—LIMITING VALUES OF x_o/t FOR SYMMETRICAL REINFORCEMENT IN BOTH FACES OF SECTION; COMPRESSION OVER ENTIRE SECTION; $n = 15$

d'/t	Values of p_o								
	0.005	0.0075	0.01	0.0125	0.015	0.0175	0.02	0.0225	0.025
0.05	0.183	0.191	0.198	0.204	0.210	0.216	0.222	0.227	0.232
0.10	0.177	0.182	0.186	0.190	0.194	0.198	0.202	0.205	0.208
0.15	0.172	0.175	0.177	0.179	0.181	0.183	0.185	0.187	0.188
0.20	0.168	0.168	0.168	0.169	0.169	0.170	0.170	0.170	0.1706

As an example of the foregoing, if the thickness of the cross-section t is 18 in., the percentage of steel reinforcement is 0.75 ($p_o = 0.0075$) and $d'/t = 0.10$, from Table 159; $x_o/t = 0.182$, and, therefore, $x_o = 0.182 \times 18 = 3.28$ in. This means that the line of pressure or point of application of the thrust cannot be more than 3.28 in. from the center line without producing tension on one side.

For convenience, the formula may be expressed as follows:

$$f_c = \frac{NK}{bt} \text{ where } K = \left[\frac{1}{1 + np_o} + \frac{x_o}{t}\,\frac{6}{1 + 12np_o\left(\frac{r}{t}\right)^2}\right]$$

Values of K, with $n = 15$ and $d'/t = 0.10$, are given in Fig. 165 for varying percentages of steel and values of x_o/t. This diagram will also serve approximately for other values of d'/t. Similar diagrams can be prepared for other values of d'/t if greater accuracy is desired, or they may be found in Hool and Johnson's "Concrete Engineers' Handbook."

To illustrate the use of the curves for K, if, in the above example, the eccentricity is 2 in., $x_o/t = 2/18 = 0.111$, and $K = 1.44$. Therefore, $f_c = 1.44N/(12 \times 18)$, from which the value of f_c can be found if the thrust N is known.

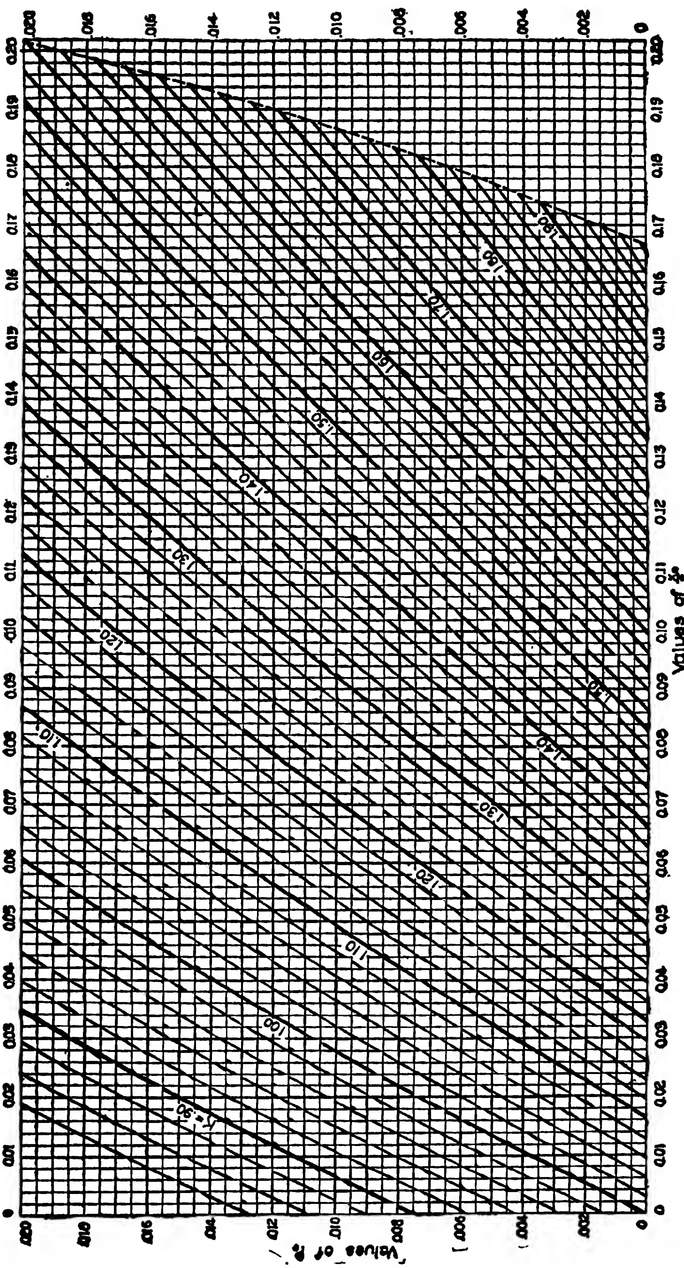
If tension does not exist in the section, the principal stress to be determined is the maximum compression in the concrete, which must not exceed a safe working stress.

Tension in Section.—When the ratio x_o/t is greater than the value given in Table 159 and the concrete is considered as unable to carry tension, the following formula should be used (see Fig. 166):

$$f_c = \frac{M}{Lbt^2}$$

where

$$L = \left[\frac{np_o}{k}\left(\frac{r}{t}\right)^2 + \frac{k}{12}(3 - 2k)\right]$$



$d' = 0.10t$ Based on $n = 15$ and $A'_s = A_s$ Values of K in formula $f_c = \frac{NK}{bt}$

Fig. 165.—Bending and direct stress—compression over whole section—symmetrical reinforcement.

To facilitate the computations, Figs. 167 and 168 are given. Determine x_o/t, and from Fig. 167 find the corresponding value of k for the given percentage of steel. Then with this value of k use Fig. 168 to find the corresponding value of L for the given percentage of steel.

For example, if the thickness of the arch, t, is 18 in., the value of $p_o = 0.0075$, x_o is 10 in., $x_o/t = 0.555$, and $d' = 0.10t$, then from Fig. 167, $k = 0.44$.

From Fig. 168, with $k = 0.44$ and $p_o = 0.0075$, $L = 0.118$

Then

$$f_c = \frac{M}{0.118 \times 12 \times 18 \times 18}$$

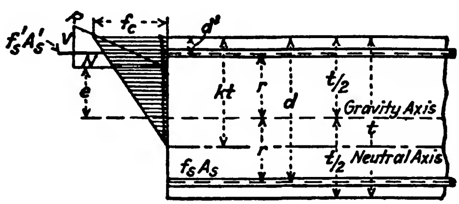


FIG. 166.—Stresses caused by a force producing compression and tension upon a reinforced section, tensile strength of concrete neglected.

from which the value of f_c can be found if the bending moment M is known.

If the value of d'/t differs materially from 0.10, some error will result from the use of these diagrams in determining the values of k. Similar diagrams for $d' = 0.05t$ and $d' = 0.15t$ will be found in Hool and Johnson's "Concrete Engineers' Handbook," as well as diagrams for $n = 12$.

Values of L, if $n = 15$, can be found as follows:

For $d' = 0.05t$, divide p_o by 0.790 before using Fig. 168.
For $d' = 0.15t$, divide p_o by 1.306 before using Fig. 168.
For $d' = 0.20t$, divide p_o by 1.778 before using Fig. 168.

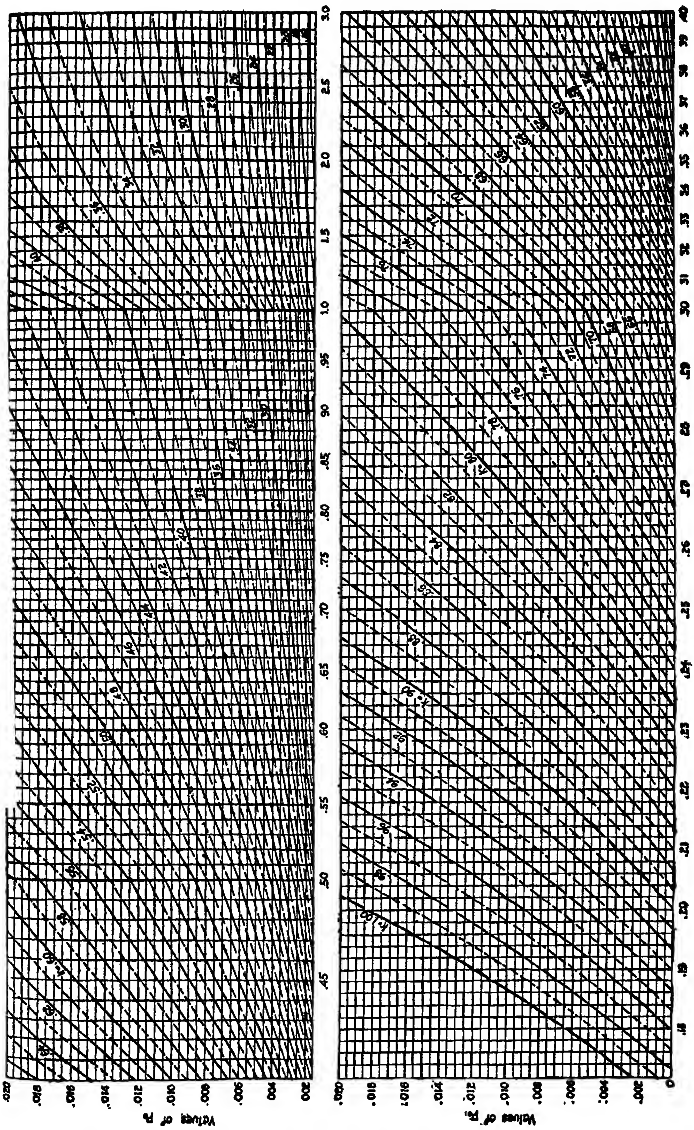
The factor with which to divide p_o for any value of d'/t is $0.16(t/r)^2$. If $n = 12$, similar methods can be used.

In the formula for determining f_c, it should be borne in mind that if b and t are in inches, the value of M should be in inch-pounds.

Having thus found the unit stress in the concrete, the unit stresses in the steel may be found by the following formulas (see Fig. 166):

$$f_s' = nf_c\left(1 - \frac{d'}{kt}\right) = \text{maximum unit compressive stress in steel.}$$

$$f_s = nf_c\left(\frac{d - kt}{kt}\right) = \text{maximum unit tensile stress in steel.}$$

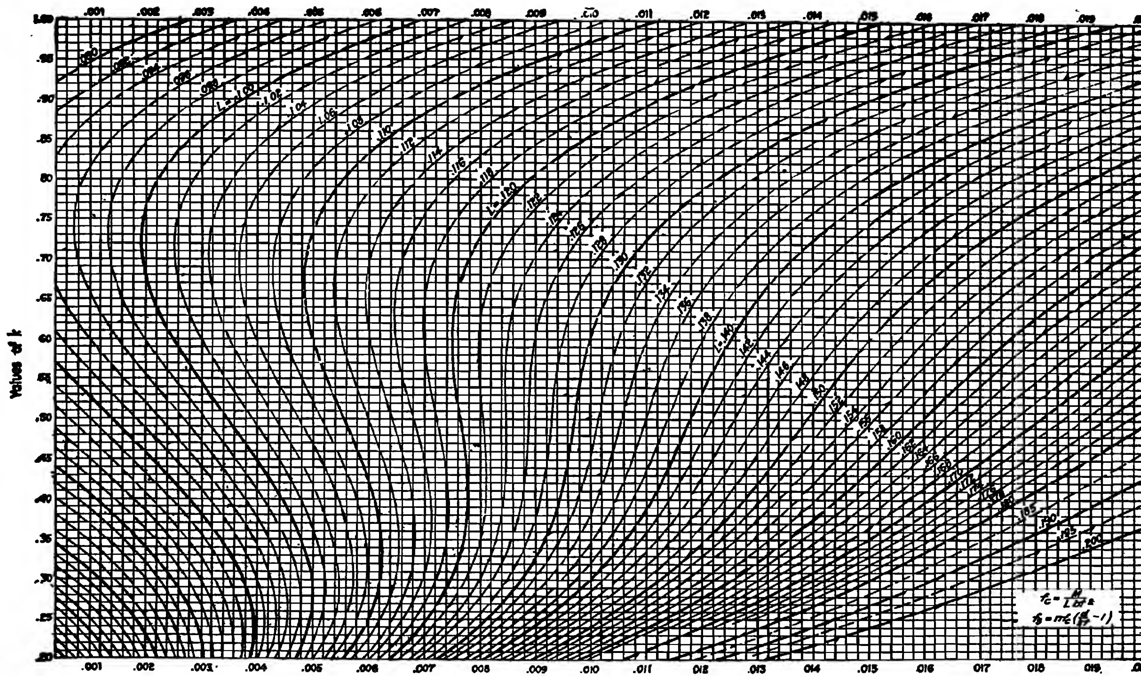


Values of $\frac{2e}{t}$

Values of k

$d' = 0.10t$ $n = 15$ $A'_s = A_s$

Fig. 167.—Bending and direct stress—tension over part of section—symmetrical reinforcement.



Values of p

$d' = 0.10t$ Based on $n = 15$ and $A_s' = A_s$ Values of L

FIG. 168.—Bending and direct stress—tension over part of section—symmetrical reinforcement.

For $d' = 0.05t$, divide p_0 by 0.790 and find value of L from above diagram.
For $d' = 0.15t$, divide p_0 by 1.306 and find value of L from above diagram.
For $d' = 0.20t$, divide p_0 by 1 778 and find value of L from above diagram.

Shearing Stress.—This is found as follows:

Let V = total shear at any voussoir.
v = maximum unit shearing stress.
b = length of arch section used, assumed as 1 ft.
jd = arm of resisting couple = approximate $\frac{7}{8}d$,
then $v = V/bjd$ = (approximate) $= 8V/7bd$.

Bond Stress.—This is computed as follows:

Let u = unit bond stress between concrete and steel bars,
o = perimeter of one bar,
Σo = sum of perimeters of bars in unit length, tension steel only,
then $u = V/\Sigma ojd$ = (approximate) $= 8V/7\Sigma od$

Unsymmetrical Reinforcement.—The discussion thus far applies only to sections reinforced symmetrically at both faces. If the reinforcement is unsymmetrical or if only the tension face is reinforced, different formulas must be used. For the methods to be used in these cases, the reader is referred to the work of Hool and Kinne[1] and Hool and Johnson.[2]

TRANSVERSE STEEL REINFORCEMENT

The fact has already been pointed out that the introduction of steel reinforcing bars to strengthen the arch where only compressive stresses exist does not permit of any great diminution of the concrete section or any marked economy, but it does have the great advantage of making the structure more reliable and acts as a sort of insurance against unforeseen stresses that may occur, such as stresses due to temperature changes or shrinkage of the concrete, settlement of foundations, and the like. It also provides an additional factor of safety against poor workmanship in the construction of the sewer section. While the designer may make an effort to foresee the conditions and to provide sufficient reinforcement or thickness of masonry to withstand the stresses as computed, there is an uncertainty concerning the action of arches for which it is impossible wholly to provide.

In view of these considerations, it is well to use transverse steel reinforcement for large concrete sewers, even though the computations may show that the line of resistance lies everywhere within the middle third of the masonry section. It is impossible in arch reinforcement to make use of the steel to the full allowable compressive working stress used in steel design. The maximum compressive stress which can be reached in a reinforced-concrete arch, designed in accordance with the foregoing method of computation, will never be greater than the allowable working

[1] Hool and Kinne, Structural Members and Connections, pp. 528–540; First Edition, 1923.

[2] Hool and Johnson, Concrete Engineers' Handbook, p. 403–406, First Edition, 1918.

TABLE 160.—COMPUTATIONS OF STRESSES

Analysis of 15-ft. 6-in. semi-elliptical sewer by method for indeterminate structures

Voussoir number	Eccentricity (ft.)[1], x_o	Thickness (ft.)[1], t	$\frac{x_o}{t}$	Area of concrete (sq. in.), A_c	Assumed steel reinforcing: Size of bars	Assumed steel reinforcing: Spacing (inches)	Assumed steel reinforcing: Area of steel both faces (sq. in. per ft.) $2A_s$	Ratio of area of steel to area of concrete p_o	k	Moment (ft. lb.)[1], M	$\frac{M}{bt^2}$ (M in. in. lb. bt in inches)	Correction factor $\cdot 16\left(\frac{t}{r}\right)^2$	Corrected p_1 for use in Fig. 169 = p_o of column 9 ÷ value in column 13	L or K	Unit compression in concrete $\frac{M}{Lbt^2}$ (lb. per sq. in.), f_c	$\left(\frac{d}{kt} - 1\right)$	Unit tension in steel $nf_c\left(\frac{d}{kt} - 1\right) \cdot n = 15$ (lb. per sq. in.), f_s	Shear (lb.), V	Unit shearing stress (lb. per sq. in.), v	Unit bond stress (lb. per sq. in.), u
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21
Crown	0 631	1.30	0.485	187 2	½" round	6	0 784	00419	431	7,100	29 2	1 173	00357	0965	305	1.017	4,650	0		
1	0 722	1 30	0 555	187 2	½" round	6	0 784	00419	390	8,563	35 2	1 173	.00357	094	374	1.228	6,885	3,950	28	127
2	0 363	1.30	0 279	187.2	½" round	6	0 784	00419	729	5,293	21 7	1 173	.00357	1055	206	0.193	600	3,900	28	125
3	0.145	1.30	0.112	187.2	½" round	6	0.784	.00419	....	2,494	. . .	......	......	1.542	142 }	$f_c = \frac{NK}{bt}$	......	3,630	26	
4	0.143	1.39	0.103	200 0	½" round	6	0 784	00392		2,832		..	...	1 470	145 }		.....	5,250	34	
5	0 491	1.51	0.325	217 2	¾" round	6	1.767	00812	688	10,876	33 1	1.055	.0077	120	276	0 290	1,200	6,140	36	111
6	0.858	1.68	0.511	242 0	¾" round	6	1 767	00731	474	20,860	51 3	1 006	.00726	.1178	436	0 896	5,860	6,580	35	106
7	1 188	1.86	0.639	267 9	¾" round	6	1 767	00660	401	30,997	62 2	958	00689	.1148	542	1 265	10,280	5,800	27	83
8	1 498	2 00	0 749	288 0	¾" round	6	1 767	00614	361	40,303	70 0	929	00661	.1125	622	1 534	14,310	6,350	28	84
9	1.966	1 94	1 014	279 4	1" square	5¾	4 175	01494	412	26,380	48 7	.941	.01587	.1675	291	1 214	5,300	16,350	73	126
10	0 415	1 94	0 214	279.4	1" square	5¾	4 175	01494	954	4 334	8 0	941	01587	1263	63	0 0		12,750	57	
11	3 405	1 94	1 755	279 4	1" square	5¾	4 175	01494	363	28,426	52 4	941	.01587	1740	302	1 513	6,860	8,550	38	66
12	6 480	1 94	3 34	279 4	1" square	5¾	4 175	01494	332	44,716	82 5	941	01587	1797	459	1 748	12,040	4,300	19	34
13	8.475	1 94	4 37	279 4	1" square	5¾	4 175	01494	324	53,063	98 0	.941	.01587	1806	542	1.815	14,750			
℄																				
Inv.	8.500	1 94	4 38	279 4	1" square	5¾	4 175	01494	325	53,267	98 3	941	01587	1804	545	1 807	14,770			

[1] These columns are in feet as they are taken from previous tables dealing with external loads, in which the unit used is the foot.

stress in the concrete multiplied by the ratio of the moduli of elasticity n. This, under ordinary conditions, places a limit in compression on the steel reinforcement of approximately 9,750 lb. per square inch (650 × 15). If a greater compressive stress should be developed in the steel, the deformation would be sufficiently great to overstress the concrete.

In good practice, the amount of transverse reinforcement in arches usually varies from about 0.2 to 1.5 per cent of the area of the concrete masonry at the crown.

In designing the reinforcement for a sewer arch, it is necessary to assume a certain percentage of steel at the start, as will be noticed from

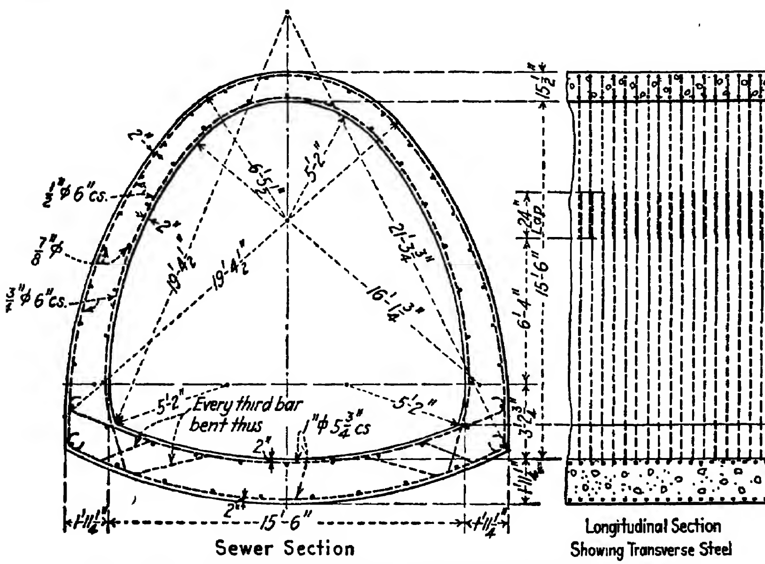


FIG. 169.—Steel reinforcement of 15½ ft. semielliptical sewer.

the method of computing fiber stresses, already given. After the computations have been made, the actual percentage to be used can be adjusted in accordance with the results of the computation, in order to obtain the most economical arrangement possible.

Computation of Transverse Reinforcement for 15-ft. 6-in. Semielliptical Section.—As an example of the method of computing the reinforcement, the following computations (Table 160) made for the 15-ft. 6-in. semielliptical section, previously analyzed, are given. As a rule, it is not necessary to compute the stresses for each division but merely for a few critical points.

It is customary to keep the same size of bars and the same spacing in the upper part of the arch, changing either or both if necessary in the

side walls or in the invert. It is desirable to have as few different sizes of bars as practicable. In general, smaller bars closely spaced are preferable to larger bars with wide spacing. A typical arrangement of the transverse steel reinforcing bars is shown for the 15-ft. 6-in. semielliptical sewer in Fig. 169.

The unit shearing stresses in voussoirs 9 and 10 (without web reinforcement) are higher than good practice allows. To insure the concrete against cracking due to excessive shearing stresses, web bars are provided to carry a portion of the shear. The masonry could have been made thicker in order to reduce the shearing stresses.

The unit bond stress is such that anchorage is required for the arch and invert bars. Such anchorage is provided by the length of bars passing through that portion of the arch where there is compression over the entire section or by hooking the bars at the ends.

LONGITUDINAL STEEL REINFORCEMENT

Masonry structures of all kinds expand and contract with temperature changes. Concrete conduits or sewers are subject to temperature changes, particularly during the period of construction. The expansion of the masonry rarely causes trouble except at sharp angles, but contraction is more likely to cause difficulties.

Cut-and-cover aqueducts have generally not been reinforced longitudinally. Transverse cracks appeared in the Wachusett and Weston aqueducts at dividing planes between day's work.[1] In the Catskill aqueduct, transverse cracks were observed in a few 75-ft. sections but none in 60-ft. sections. No cracks wider than 0.04 in. were found in the Wachusett aqueduct. The aqueduct connecting the Jerome Park reservoir (N. Y.) with the new Croton aqueduct was built with no allowance for temperature stresses, and only a few trivial transverse cracks developed. No expansion joints were provided in the Los Angeles aqueduct from Owens River or in the Winnepeg aqueduct. There is ample precedent, therefore, for omitting reinforcement against temperature changes. If this is done, it may be necessary to repair some cracks, particularly if the invert, side walls, and arch are built separately with construction joints not in the same plane.

The actual amount of steel reinforcement to be provided to resist temperature stresses is, to a certain extent, a matter of judgment. For a sewer constructed in comparatively dry soil and designed to carry both surface water and sewage, the presence of small cracks might be considered unobjectionable. Large cracks would be objectionable on account of the possible rusting of the steel reinforcement and consequent weakening of the structure. For a sewer constructed in very wet soil adjacent to a river or a creek, where it is essential to keep out as

[1] Flinn, Weston, and Bogert, "Waterworks Handbook," First Edition, 272.

much ground water as possible, the presence of even small cracks might be objectionable.

It is interesting to note that concrete laid during warm weather is much more likely to crack on account of temperature changes than concrete laid during cold weather, and, in addition, shrinkage cracks are more apt to occur with concrete laid during hot, dry weather unless care is taken to keep the concrete wet.

Two methods are in use for preventing objectionable cracks caused by the shrinkage of concrete in hardening and the contraction due to a lowering of the temperature. One method is to locate expansion joints at frequent intervals, approximately 30 ft., so that all of the changes will be concentrated in one crack at each expansion joint. The second method is to insert enough reinforcement composed of small bars placed near the surface of the concrete to distribute the cracks at short intervals and make them so small as to be practically invisible or unobjectionable. In actual practice, it has been customary to insert from 0.2 to 0.4 per cent of the area of the concrete as longitudinal steel to resist shrinkage and temperature stresses. For this purpose, deformed bars furnishing a high mechanical bond with a high elastic limit are advantageous.

To illustrate the action of steel in resisting the contraction due to temperature changes, consider a cylinder of concrete having a cross-section of 100 sq. in. and having a steel rod passing through it from end to end and fixed to anchorages immediately beyond the ends of the concrete cylinder. If the rod were independent of the concrete and were subjected to a drop in temperature of 10°, it would tend to contract an amount equal to the product of its length, the fall in temperature and the coefficient of contraction or

$$\begin{aligned}\text{Contraction} &= L \times 10 \times 0.00,000,55 \\ &= 0.00,005,5L.\end{aligned}$$

The stress developed in the steel because of its fixed ends is the product of the contraction per unit of length and the modulus of elasticity of steel, or,

$$\begin{aligned}\text{Stress} &= 0.00,005,5 \times 30,000,000 \\ &= 1,650 \text{ lb. per square inch.}\end{aligned}$$

Similarly, if the concrete cylinder were independent of the steel and were fixed at the ends, the stress developed would be

$$\begin{aligned}\text{Stress} &= 10 \times 0.00,000,55 \times 2,000,000 \\ &= 110 \text{ lb. per square inch (11,000 lb. in 100 sq. in.)}\end{aligned}$$

The steel rod is attached to the concrete through the bond between concrete and steel surface, and ultimately transmits to the cylinder the stress to which it would be subjected if its ends were fixed. The distance necessary for this stress of 11,000 lb. to be communicated from

the steel to the concrete depends on the size of the rod and the bond strength. If the rod is ½ in. round, it will have a cross-sectional area of 0.20 sq. in. and a bond surface of 1.57 sq. in. per inch of length. The ultimate bond strength for plain steel rods is from 200 to 300 lb. per square inch. Assuming the higher value, the bond strength per inch of rod is

$$1.57 \times 300 = 471 \text{ lb. per inch.}$$

The distance required to transmit the stress from concrete to steel is

$$\frac{11{,}000}{471} = 23 \text{ in.}$$

The stress in the steel due to its change in temperature is

$$1{,}650 \times 0.20 = 330 \text{ lb.}$$

The stress in the steel at the anchorages is 11,330 lb.,[1] the sum of the stress due to the change in temperature in the steel and that transmitted to the steel from the concrete. The stress in the concrete increases from zero at the ends of the cylinder to 11,000 lb. at points 23 in. from the ends and remains uniform between these points. The stress in the steel decreases from 11,330 to 330 lb. and is this amount between the points 23 in. from the ends.

If the drop in temperature be 18°, the stress in the concrete will be 18 × 0.00,000,55 × 2,000,000 = 198 lb. per sq. in. or practically 200 lb., which is a high value for the ultimate tensile strength of 1:2:4 concrete and the concrete will yield by cracking.

The total stress in the concrete is 200 × 100 = 20,000 lb.

The length necessary for this stress to be communicated to the steel is

$$\frac{20{,}000}{471} = 42 \text{ in.}$$

The stress in the steel due to the change in temperature is 18 × 0.00,000,55 × 30,000,000 × 0.20 = 594 lb.

The maximum stress in the steel, including that communicated to it from the concrete, is

$$20{,}000 + 594 = 20{,}594 \text{ lb.}^1$$

The stress in the concrete is uniform and equal to its ultimate strength for a considerable length between the points at which the stress due to the contraction of the concrete begins to be transmitted to the steel.

The first crack will occur at the weakest point in this length. It may be assumed that the first crack will occur 42 in. from one end, say the left; it will be immediately followed by a second crack 42 in. from the first, and so on, until the distance from the last crack to the right

[1] The corresponding unit stress in the steel is excessive, showing that the relative amounts of steel and concrete assumed for the example would be incorrect for construction.

end is less than $2 \times 42 = 84$ in., when no more cracks will form, since the stress in the concrete will nowhere reach 20,000 lb.

If this length (say 83 in.) of cylinder were not restrained by the steel, it would contract $18 \times 83 \times 0.00,000,55 = 0.0082$ in., and this would be the maximum width of crack. As, however, the bond strength throughout the length of the section between cracks is sufficient to resist a part of the tendency to contract, the width of the crack will be somewhat less than this amount.

The distance between cracks is inversely proportional to the bond stress developed. For this reason, it is well to use small bars for temperature reinforcement in order to obtain a large bond surface. It is also preferable to use deformed bars, the unit bond strength of which may be taken as 25 per cent higher than that of plain bars, in order better to ensure obtaining a maximum bond strength of about 300 lb. per square inch.

The amount of steel reinforcement required will depend on the range of temperature to be provided for. It is the usual practice to allow the temperature steel to be stressed to near its elastic limit. The amount of steel to be provided will be, therefore, that which will sustain the stresses in the steel itself with sufficient margin to ensure the rupture of the concrete.

Assuming a drop in temperature of 35° and an elastic limit of 50,000 lb. per square inch for a hard grade of steel, the amount of reinforcement may be computed as follows:

Stress due to temperature change in steel itself

$$= 35 \times 0.00,000,55 \times 30,000,000 = 5,775 \text{ lb. per square inch.}$$

Stress which can be transmitted to concrete

$$= 50,000 - 5,775 = 44,225 \text{ lb. per square inch.}$$

Cross-section of concrete which can be ruptured using 200 lb. per square inch as the ultimate tensile strength of concrete

$$= \frac{44,225}{200} = 221 \text{ sq. in. of concrete per square inch of steel.}$$

$$\text{Percentage of steel} = \frac{100}{221} = 0.45.$$

In practice, 0.4 per cent of steel has generally been found adequate reinforcement against temperature stresses.

SAFE WORKING STRESSES

The working stresses recommended by the Joint Committee on Standard Specifications for Concrete and Reinforced Concrete[1] furnish the best guide for determining safe values to be used in design. For a

[1] *Proc. Am. Soc. C. E.*, October, 1924; 1209. Also published in the Proceedings of the American Concrete Institute.

complete understanding of the following figures, reference should be made to that report.

The following maximum allowable working stresses for concrete are stated in terms of f'_c, the ultimate compressive strength of the concrete at age of 28 days, based on certain prescribed tests. Values are shown for an assumed ultimate compressive strength of 1,625 lb. per square inch which is a safe allowance for concrete composed of 1 part of Portland cement and 6 parts of aggregate placed under the ordinary conditions of sewer construction. The strength assumed in design, however, should be chosen to meet the conditions likely to be encountered.

		Pounds per square inch
Compression in extreme fibre[1]	$0.40f'_c$	650
Shear in beams without web reinforcement, straight bars without special anchorage	$0.02f'_c$	32.5
Shear in beams with straight bars with special anchorage	$0.03f'_c$	48.75
Shear in beams with stirrups or bent-up bars or combination of the two, straight bars without special anchorage	$0.06f'_c$	97.5
Bond stress for ordinary anchorage in beams		
for plain bars	$0.04f'_c$	65.0
for deformed bars	$0.05f'_c$	81.25
Tension in billet steel reinforcing bars, structural steel grade	....	16,000

[1] NOTE: Do not confuse the symbol f'_c of this discussion with f_c under "Computation of Stresses in Arch Section," as it is used in two distinctly different senses.

If "special anchorage" is provided in the tension reinforcement to meet the requirements of the Joint Committee, the allowable shear and bond stresses may be greatly increased.

CHAPTER XV

INLETS, CATCHBASINS AND MANHOLES

The special structures which are built on sewerage systems have an important part to play in the operation of such works, as a rule. In order to clean sewers, manholes giving access to them are provided, and drop manholes and wellholes have been developed from ordinary manholes, in order that sewage may be delivered vertically from one elevation to another with a minimum amount of disturbance. For this latter purpose flight sewers, with their inverts like a straight stairway, have also been constructed. Where storm water is removed underground, street inlets are provided to discharge it directly into the sewers and drains, and catchbasins are employed where this surface runoff contains so much refuse of different kinds that the engineer prefers to give it a chance to settle in a readily cleaned sump rather than to allow everything to flow without check into the sewers. In order that long lines of small sewers may be kept under observation with the greatest facility, some engineers provide them with lampholes, down which a lamp can be lowered to illuminate the interior of the sewer enough to enable an observer at the manhole on either side of the lamphole to see with more or less distinctness the condition of the pipe.

There are many small sewers with grades so flat that the only way to keep them clean is to flush them with water, accompanied, if necessary, by scrubbing with a brush on the end of a long jointed rod or wire. For this purpose, a flushing manhole operated manually or an automatic flushtank is employed, and there is a great difference of opinion among engineers regarding the respective merits of the two types. Occasionally a flushing inlet is provided on the bank of some river or pond, through which water can be admitted to large sewers which need cleaning.

Where large sewers join there are bellmouths and other forms of junctions to be built, which sometimes assume forms of considerable complexity. Inverted siphons are used in crossing valleys or dropping below subways and other obstructions. On rare occasions a true siphon may be used to overcome a small ridge, although it is usually considered preferable to go to considerable expense to avoid such a detail. Since reinforced concrete came into use, specially designed hollow girders or beams have been employed in some places to cross rivers or deep gulches,

where inverted siphons or steel bridges would have been used before. If the combined sewerage system includes intercepting and relief sewers, some form of regulating device must be used at each place where the sewage is discharged from a collecting sewer into an intercepting or relief sewer; there are numerous forms of automatic regulators, storm overflow chambers, and leaping weirs used for such situations.

Where the sewage is discharged into a river, lake, or tide water, an outlet of some kind is needed; it has already been pointed out in the Introduction that the failure of the designers of early sewerage systems to allow for the effect of tide-locking of such outfalls caused a large part of the really serious troubles with some of the sewerage systems built prior to about 1875. Even today the effect of submergence on the flow in an outfall sewer and on the discharge from its outlet is not always given the attention it requires. Another allied type of special structure is the tide gate, which is a large check valve to prevent the entrance of water into a sewer when its surface elevation reaches such a height that the water tends to pass in through the valve rather than the sewage to pass out.

In the early days of sewerage works, their ventilation received much attention and a variety of theories existed concerning the best way to carry this out. The omission of the main house trap was advocated by some engineers as a material aid in sewer ventilation, because of the upward draft through the soil pipes of the buildings which, it was claimed, would come into existence in this way. Another body of engineers vigorously opposed the omission of the main trap and insisted upon a vent pipe run from the house drain, outside the trap, up the side of the building to an outlet above the highest windows. Still other engineers made use of ventilating chimneys shaped like the posts of street lamps, and sometimes used as such, and, at one time, perforated manhole covers were in quite general use as a means of ventilation. Taking it all in all, it is perhaps safe to say that there has been no part of sewerage engineering in which a greater variety of special designs has been prepared for the same purpose than in ventilation, while the vigor of the debates over it down to the last decade of the last century was a noteworthy feature in the engineering literature of the day. It was at one time thought that sewers should be ventilated at manholes so as to prevent sewer gases from accumulating and finding their way into dwellings, affecting the health of the residents. The real need for ventilation, however, arises from the danger to workmen in sewers and manholes. Sewer gases have caused fatalities on several occasions, usually where agitation of the sewage caused the release of excessive quantities of gas. The removal of volatile gases by ventilation is an aid in reducing the danger from explosions.

Although some of these special structures offer little opportunity for standardization, due to differences in local conditions, in each class there are certain features which experience has indicated are important. During the past few years, there has been an attempt at standardization both as to types and details of some of these special structures. This has been a slow process as experience rather than theory has been the controlling factor. Such attempts toward a common practice as have been made will be described under the appropriate headings.

STREET INLETS

That part of the storm water termed runoff, which is not lost by evaporation or percolation, drains into the street gutters to be removed at suitable intervals through inlets connecting with the sewerage or underground drainage system. These inlets either discharge directly into the drains or into catchbasins which are intended to intercept the refuse and sediment flushed from the street surfaces. Their location is a matter in which the street department is also interested, as it is important that both pedestrian and vehicular traffic be interfered with as little as possible. While it is not practicable to lay down instructions which will cover all cases, certain general rules which are usually applicable may be given.

Where cross streets are not too far apart, inlets are generally located at street intersections in such a way as to intercept water flowing in the gutters before it reaches the crossings used by pedestrians. Where the distance between intersecting streets exceeds 300 to 500 ft., the water may accumulate to a sufficient depth in the gutters to interfere with passing vehicles. In such cases it is better to construct inlets at intermediate points to prevent this condition from occurring. Also where the slope of the gutter is steep, intermediate inlets should be placed at suitable points to assure a rapid removal of the stormwater. In such case, it is desirable to depress the gutter several inches at the inlet, bringing it rather abruptly to grade again beyond the inlet, in order to intercept the water which has sufficient normal gutter velocity to pass by the opening. If permissible, the depression should be carried some distance out from the curb, and changes of slope should not be so abrupt as to cause inconvenience to vehicular traffic. Such depressions are not looked upon with favor by street departments, but are desirable from the standpoint of drainage. It is the sudden change in slope rather than total depression of the gutter that is objectionable, and with due care adequate drainage and convenience for traffic both can be obtained. For hillside inlets it may be necessary to increase the length of opening in the curb to provide sufficient capacity. Figure 170, reproduced from "City Pavements" by Major F. S. Besson, is an illustration of such an elongated inlet in the curb.

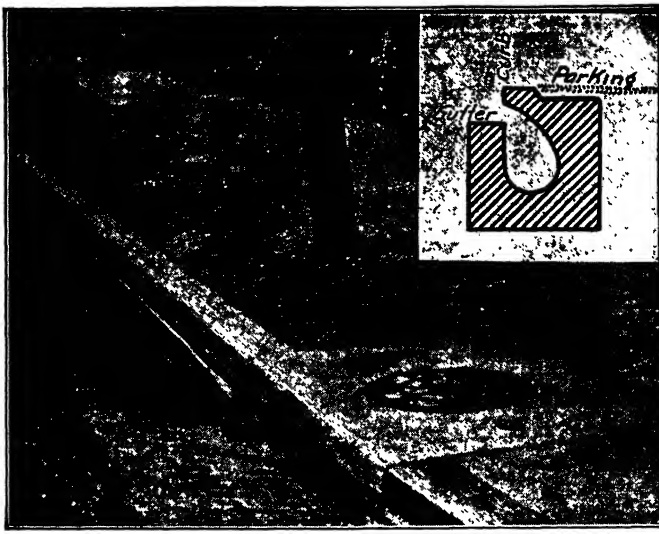


FIG. 170.—An elongated curb inlet on a steep grade. This construction is used in order to catch the water without the use of an objectionable obstruction in the gutter.

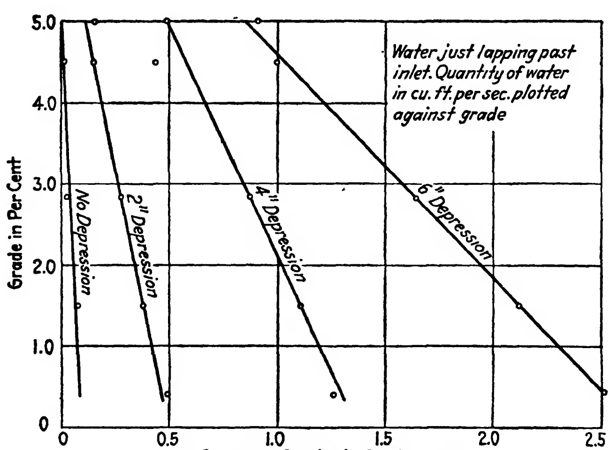


FIG. 171.—Relation between street grade and quantity of water taken through inlets, for various depths of depression (St. Louis).

Inlet opening is 4 ft. 6 in. long and 6 in. high.

There are few data available on the carrying capacity, especially on grades, of curb openings which are usually from 2 ft. 6 in. to 4 ft. 0 in. long and 6 to 8 in. high.

Tests to determine the capacity of inlets and the effect of longitudinal grade in the gutter and of depression of the inlet, were made by W. W. Horner of St. Louis.[1] The best type of inlet tested had a clear opening of 6 in. by 4½ ft. The sill sloped abruptly down from the gutter and there was no grating in the gutter. The computed intake capacity of this inlet was 1.0 cu. ft. per second. Figure 171 shows the measured capacity of the inlet with water just lapping past, and for various street grades. For larger discharges in the gutter, the inlet will take greater quantities of water but some will pass on down the gutter. The grades and depths of sump or depression are shown. It will be noted that to take 1 cu. ft. per second requires a 4-in. depression on a grade not to exceed 2 per cent, or a 6-in. sump on grades up to 4½ per cent. Horner says that

. . . the converging lines seem to indicate that with the new style single inlet it would be impossible to take in water on grades of over 7 per cent and that this condition is certainly true for any of the sump depths within the range of this test, and within the range of probability. This indicates, therefore, that *for all steep grades a different type of inlet is required* and an inlet with a long opening . . seems desirable.

He has stated elsewhere,

I think the use of standard inlets at standard locations, without regard to the work required of them, is the most common fault in sewer design. The street pavement officials usually demand that there shall be no break in the curb line at an inlet and no great depression in the pavement or gutter. Under these conditions inlets on steep streets, other than those at the foot of the grade, are almost useless. Our standard opening in the curb is 4 ft. long and 8 in. high, and only a small proportion of the height, 2 to 4 in., is below the normal gutter line. The only solution seems to lie in the multiplication of these openings, two to four in a series, and a continuous basin behind the curb and under the sidewalk.

Although inlets frequently are located at the angle of intersection of two streets, this is not considered the best practice. At this location, they are in the line of heaviest travel which may injure both the inlet casting and adjacent pavement, while this position is also unfavorable for the rapid removal of the storm water. The resultant direction of flow of the intersecting streams is at this point away from the inlet rather than toward it. The better arrangement is to place inlets above each crossing.

[1] *Munic. & County Eng.*, 1919, 57, 147.

In case of doubt it is well to remember that the convenience of the public is better served by having too many rather than too few inlets. What has been said applies equally well to street inlets and to catch-basins, although there is considerable difference between these two classes of structures.

Since a street inlet affords a direct connection between the gutter and the sewer, it is very important that it should be so designed that as little opportunity as possible exists for its stoppage. The obstruction may arise through the clogging of the opening (mouth or gully) by

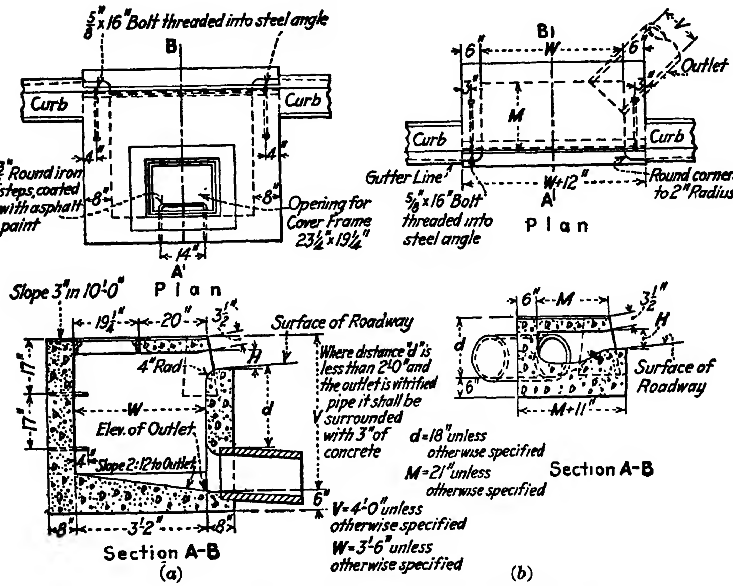


FIG. 172.—Standard curb inlet, Los Angeles, Calif.

which the water enters, or it may occur in the trap if it has one, or in the pipe running to the sewer. The objects which cause the most trouble at the openings of the inlet are sticks, waste paper, and leaves. If sticks become lodged against the opening, the leaves and waste paper drawn to it by the next flush of storm water are likely to cause a stoppage. To avoid this some engineers have tried the use of openings presenting the least possible obstacle to the entrance of these three classes of refuse.

Where there is much waste paper or leaves, a grating placed in the gutter may be of little value in removing storm water as it may be closed either partially or completely at the first runoff from the storm. It would appear advisable in most cases, therefore, to provide an opening

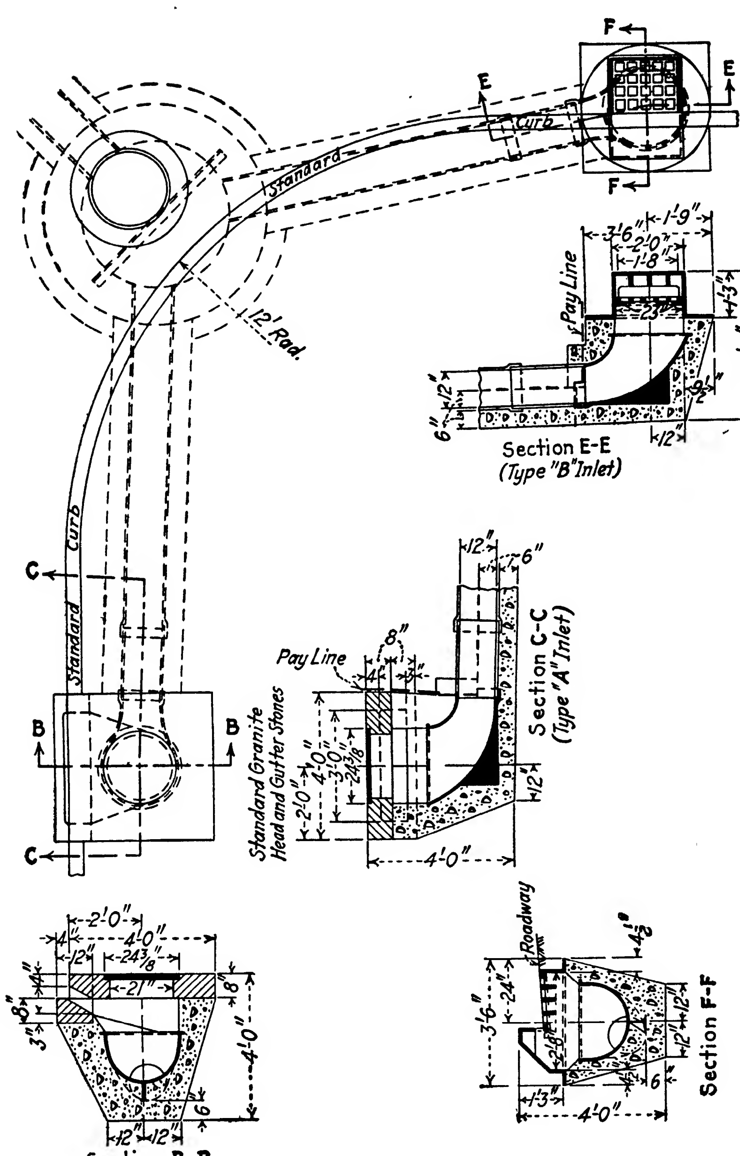


FIG. 173.—Standard inlets, Borough of Manhattan, New York City.

in the curb adequate to remove the total flow accumulating at the inlet. Cleanliness of streets and adjacent sidewalks is a great aid to the successful functioning of inlets during storms.

If the sewers in a district are on self-cleansing slopes except at a few points, it may be best to construct grit chambers in the sewers near these places in order to keep down the expense of maintenance by forcing most

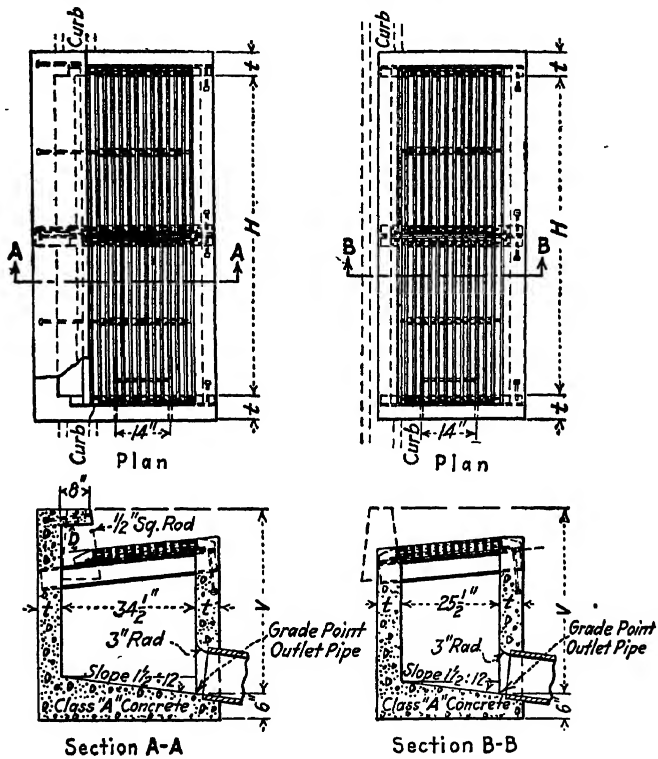


FIG. 174.—Standard inlets with gratings, Los Angeles, Calif.

of the grit to gather in pits whence its removal will be less expensive than from the sewers of flatter slopes.

While an open grating is ordinarily used in the gutter at inlets, some engineers depend entirely upon the curb opening for receiving the storm water. The Los Angeles inlets shown in Fig. 172 are of this type. The greater depth of (*a*) is due to the necessity of carrying the outlet drain beneath the pavement, which is not the case for the shallow inlet (*b*). Since all materials entering these inlets reach the sewer, self-cleansing grades must be used.

In Fig. 173 two inlets are used with a single catchbasin which is sometimes combined with one of the inlets. In this standard of the Borough of Manhattan, the Type *B* inlet is shallow and consists of a casting with a central supporting rib surmounted by an inlet casting and grate of the usual type. The Type *A* inlet has no opening in the gutter but it can be inspected or cleaned through an opening in the sidewalk closed with a standard basin cover.

Where a larger inlet opening is required, due either to the greater amount of runoff to be handled or to a steeper gutter grade, the Los

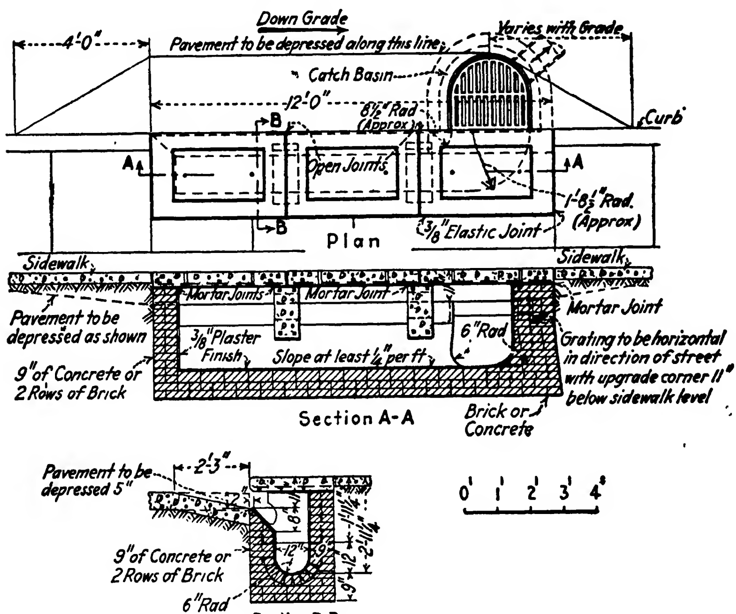


FIG. 175.—Standard multiple curb inlet, San Francisco, Calif.

Angeles inlets shown in Fig. 174 may be used, these having an assembled grating rather than the usual casting. Another method of providing a longer curb opening is by using a multiple curb inlet similar to the San Francisco standard (Fig. 175). This shows how the multiple opening in the curb may be used with a single gutter grating or *D*-frame.

CATCHBASINS

The catchbasin was formerly considered an absolutely essential part of any American combined sewerage or drainage system. Experience had shown that in many sewers the velocity of the sewage was insuffi-

cient to prevent the formation of sludge deposits, and it was manifestly more expensive to remove this sludge from the sewers than from catchbasins. This experience was gained in days when the pavements of American streets were crude and little attention was paid to keeping them clean. The sewers themselves were not laid with the present regard for self-cleansing velocities. Under such conditions it was but natural that catchbasins should find more favor than they do at the present time. Durable pavements, more efficient street cleaning, and sewers laid on steeper slopes have reduced the need for such special structures to a few situations. The following quotations show the trend of opinion at the present time.

We are also of the opinion that the inlets should not be provided with catchbasins to retain the filth or whatever may be washed into them. The object of such basins is to intercept heavy matter and periodically cart it away, instead of allowing it to reach the drains and there to deposit. Catchbasins, even after the sewage flow no longer exists in the gutters, are still apt to get foul because of the organic matter washed from the street. Such foulness is less offensive in the drains than in the catchbasins which are situated at the sidewalk and where it is much more likely to be observed. Also, it is found impracticable to intercept all matter in the catchbasins which would deposit in the drains after they reached the flat grade in the lower part of your city. The cleaning of the drains would, therefore, be necessary in any event, and the additional amount of filth that would otherwise be intercepted by the catchbasins, will not cost much more to remove.[1]

Theoretically desirable, catchbasins are, in reality, among the most useless devices employed for the removal of solid material from sewage. They are generally ineffective because they are not cleaned with sufficient frequency to enable them to serve as traps. It seems impracticable to keep them clean. To maintain catchbasins in serviceable condition requires much hand labor, and this is costly. The work is usually carried on to the annoyance of pedestrians and householders. Some sewerage systems are without catchbasins and their elimination, as a general procedure, is much to be desired.[2]

That the sewers built by the Commission might become at once effective in providing for the disposal of storm water and thus fully useful at as early a date as possible, the Commission has built 225 storm-water inlets, of which some have been in the form of catchbasins. Careful consideration was given to the desirability of building inlets rather than catchbasins, as has been the city's custom for many years. It was felt, however, that in this climate it was unwise to provide pools of water in which mosquitoes could breed, as in the case where catchbasins are built, and further that under existing conditions the catchbasins, for the detention of detritus, were not necessary in most cases. It was also found that it was already the

[1] HERING, RUDOLPH, and SAMUEL M. GRAY, *Report* for Sewerage and Drainage in Baltimore.

[2] *Report* Metropolitan Sewerage Commission, New York, 1914.

practice of the Board of Public Works to build inlets instead of catchbasins. The inlets as built have been untrapped and the experience thus far indicates that this type of inlet has given satisfaction.[1]

In rural districts the gully retainers are often allowed to stand full of grit for months together, and any such detritus brought down by the rain thus runs straight into the sewers. If the retainers are not going to be emptied after each heavy fall of rain they might as well be omitted, as they are serving no good purpose, and may even cause considerable odor when they are allowed to stand full for long periods. In other places the gullies may only have to take water flowing on large paved areas where no mineral matter of any importance can reach them. In such positions the retainer merely serves to retain soft matter which would be better in the sewers. When we remember that a velocity of flow equal to 3.3 ft. per second will carry pebbles 1½ in. in diameter along a sewer, and that a flow of 0.7 ft. per second will remove coarse sand, and that a flow of 0.5 ft. per second will remove fine sand, allowing every margin of safety, it seems that there can be very little object in taking so much trouble to exclude the washings of such paved roads. The author does not wish it to be understood that he thinks that retainers and traps are generally unnecessary, but he considers that there are very many cases in which the traps and retainers might be omitted with advantage, and in which the comparatively fine grid might be omitted in favor of a larger opening.[2]

During recent years the increasing mileage of smooth pavements and the use of gradients in storm sewers sufficient to provide self-cleansing velocities, have made flushing of streets a popular method of cleaning. Where this method of street cleaning is used, it is obviously undesirable to have catchbasins as these would quickly be filled under the usual conditions of flushing, and the remainder of the dirt would be carried into the sewer. The material accumulating in the catchbasin would also be more expensive to remove than if left in the gutter. So, since much of the solid materials will reach the sewer after the catchbasin has been filled, self-cleansing gradients are not less essential because catchbasins are employed. Many types of specially designed apparatus are now in use for flushing streets. The horse-drawn flush cart washes more of the refuse into the inlets than do the motor flushers, due to the greater quantity of low-velocity water used. With motor flushing, the greater force of the water cuts the refuse from the street surface equally well, while the smaller quantity of water is insufficient to transport all of it and leaves much of it in the gutter where it can easily be removed.[3] In some cities, as for example, Rochester, N. Y., catchbasins are not used.

The method of cleaning the catchbasin may be an important factor in its designing. In Chicago, where there are some 130,000 catchbasins and

[1] Breed, J. B. F., and Harrison P. Eddy, *Report* to Commissioners of Sewerage of Louisville, 1913.

[2] Watson, H. S., "Sewerage Systems."

[3] *Am. City*, 1922; **27**, 300.

95,000 manholes to be cleaned (1923) and where motor flushing of streets is used, the method of cleaning is by means of a bucket and hoist mounted on a truck. A small orange-peel bucket which will enter the standard catchbasin opening is used in a number of cities. Another effective method of cleaning is by an ejector or eductor mounted on a truck and operated from its motor. The catchbasin should be designed so that there shall be no interior arrangements that will be damaged in this cleaning process; and since with such a bucket, much material is spilt about the opening, the catchbasin with opening in the gutter is to be preferred to that opening in the sidewalk.

Catchbasins are certain to be used, even in well-managed cities, as receptacles for street refuse which should be gathered otherwise. This was well stated as follows, by the Metropolitan Sewerage Commission of New York in its report of 1910:

> The men of the street-cleaning department wash some of the paved streets in certain sections of the city, and during this operation much detritus is carried into the catchbasins. The custom of pushing street sweepings into the basins appears to be quite general; and, in fact, the basins seem to be popularly considered proper receptacles for anything that will enter the openings, including snow in winter. The report of the Bureau of Sewers for 1907 states that 9,674 basins were cleaned of snow. Although there is an ordinance against putting snow and street sweepings into the basins, the magistrates have invariably dismissed the cases when the street cleaners have been arrested on complaint of the Bureau of Sewers for violation of the ordinances.

While there is not the crying need for catchbasins at frequent intervals which was formerly believed to exist, they have their uses where it is probable that large quantities of grit will be washed to the inlet and if this enters the sewers it is likely to cause obstructions in them. If they are used they should be cleaned whenever necessity arises. Cleaning should not be neglected until stoppage and the attendant flooding occurs, nor should basins be cleaned where there is little accumulation in them unless in localities where the nature of the deposit is such as to create offensive odors which may escape from the basin and prove a source of annoyance to persons passing or living nearby. A basin may be put out of service automatically when it becomes filled. This is accomplished by the old-fashioned basin shown in Fig. 176 which represents a Providence structure. The feature of this catchbasin is the trap. As sediment collects in the catchbasin it reduces the space available for water above its top and below the water line established by the lip of the trap. Eventually there will be very little water capacity, and in summer, in prolonged dry weather, the water will evaporate to such an extent that odors may arise from the catchbasins. If no odors arise and the cleaning gang does not reach the basin in its regular routine, the

sediment will gradually accumulate until it overflows the edge of the trap, blocking it. When this occurs the first heavy storm will give undeniable evidence of the necessity of cleaning. In this way the trap

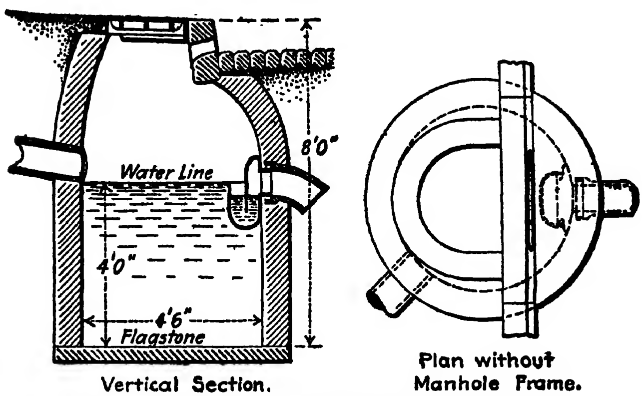


Fig. 176.—Standard catchbasin, Providence, R. I.

serves a useful purpose by preventing the escape into the sewer of large quantities of silt which might form deposits. Another advantage of this basin, due to its trap, is that the water which accumulates in it can be

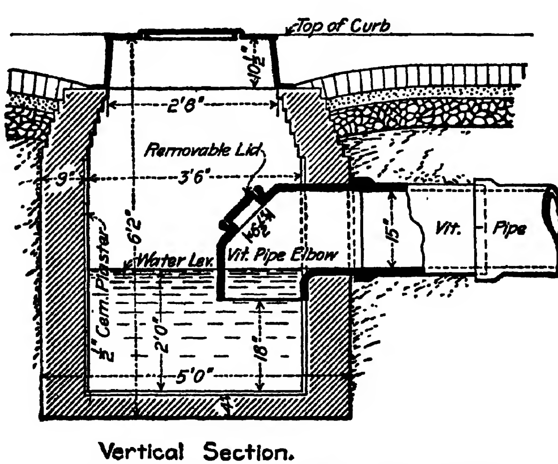


Fig. 177.—Standard catchbasin, Columbus, Ohio.

bailed by the cleaning gang into the trap and thus delivered directly into the sewer, instead of being lifted to the top and thrown over the street.

The great disadvantage of the trap is its liability to freeze in cold weather, although it should not be forgotten that the air inside the sewers, which will come up to the sewer inlet, will be somewhat warmer than the outdoor atmosphere, and the sheltered position of the trap also has some effect in reducing the danger of freezing. Where basins are connected to storm drains, there will be much greater opportunity for the freezing of traps. As in all attempts to use traps on catch basins or inlets, the

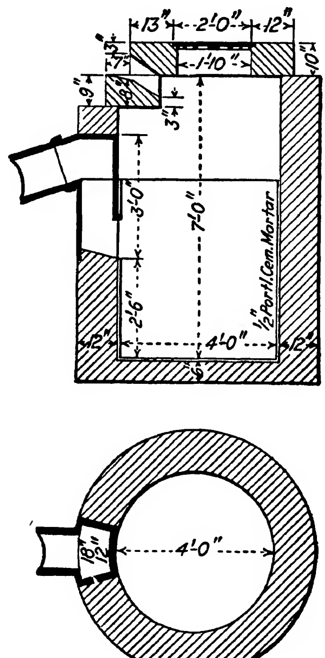


Fig. 178.—Standard catchbasin, Newark, N. J.

permanence of the water seal is very questionable. It will evaporate during prolonged dry weather, and it is idle to expect that a sewer department will keep all traps filled by means of a hose during such seasons.

The type of catchbasin used in Columbus, Ohio, for many years, is shown in Fig. 177. It has two drawbacks, both due to the use of vitrified pipe for the elbow. It is difficult to believe that such vitrified elbows will withstand the hard knocks given to them during the operation of cleaning basins. This is rough work done as expeditiously as

possible, and everything within a catchbasin should be designed to withstand hard usage. A second drawback to the basins for use in northern latitudes, is the possibility that ice will damage the elbow. The standard Newark catchbasin (Fig. 178) is typical of the form in which the trap is in the wall of the basin. The standard catchbasin of the Borough of Manhattan is shown in Fig. 179. In the two types

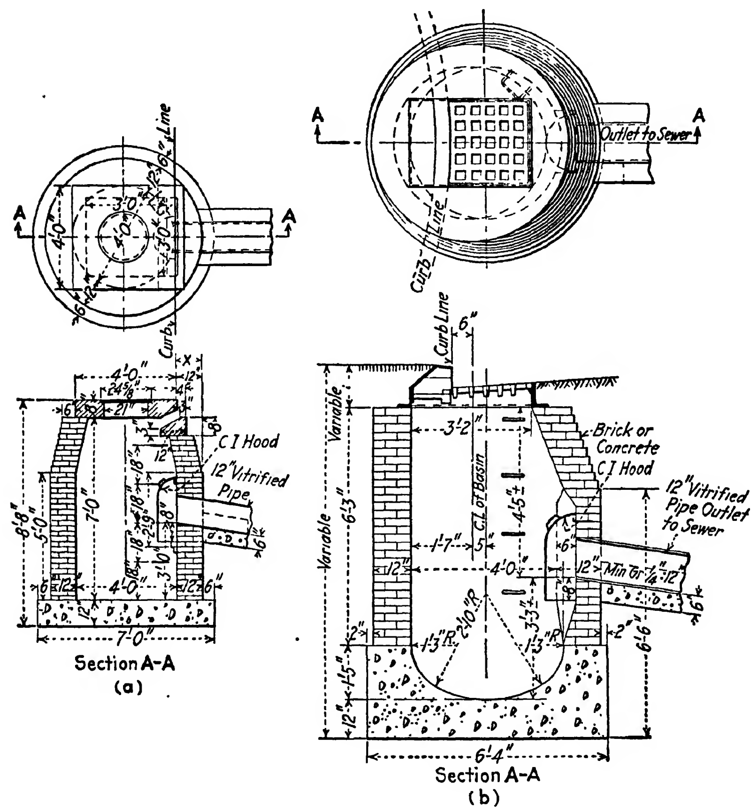


FIG. 179.—Standard catchbasin, Borough of Manhattan, New York City.

of catchbasin here shown, that in (*a*) has an opening in the sidewalk while in (*b*) it is located under the gutter with storm water admitted both through a gutter mouth and a grating in the gutter itself.

To meet the conditions at a railroad round house, where large amounts of cinders and dirt must be excluded from the drain, and where the use of a grab bucket for cleaning is especially advantageous, the Illinois Central Railroad has constructed a catchbasin having a

screen chamber adjoining, in which is an iron rack over the outlet pipe. The arched opening between the basin and the screen chamber

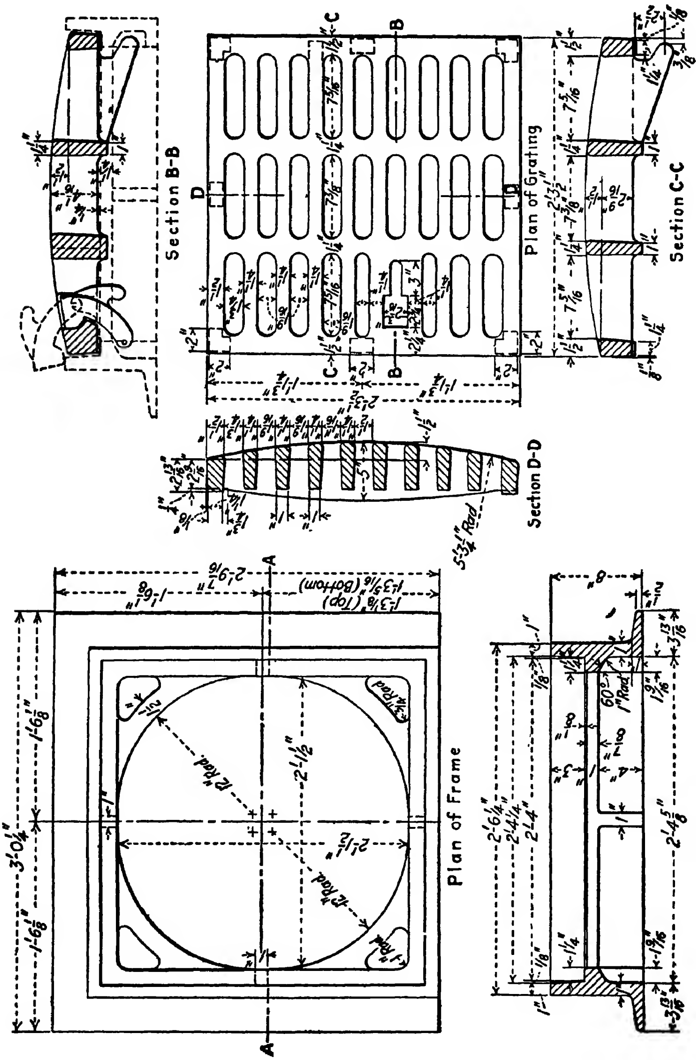


FIG. 180.—Standard inlet castings, Boston, Mass.

is below the elevation of the outlet, and forms a trap which is not subject to injury by the bucket.[1]

[1] *Eng. News-Rec.* 1928; **100**, 431.

The special catchbasins adopted to intercept gasoline and oil coming from automobiles and motor trucks are discussed in Vol. II.

The materials used for catchbasins comprise concrete, brick, and stone, the relative merits of the three for any case depending almost entirely on the cost of the finished structure, since good basins can be constructed of any one of them. This feature of the subject is discussed in greater detail in the chapter on the construction of masonry sewers in Vol. II.

Castings.—For many years a great variety of castings for catchbasins and inlets have been used. The present tendency, however, is toward

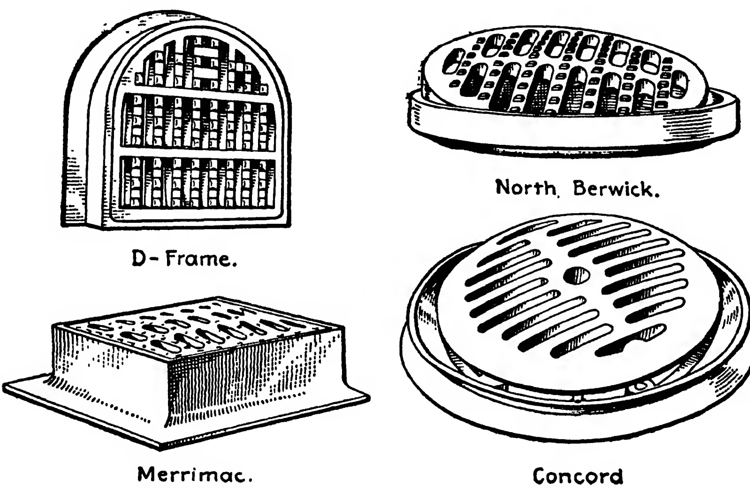


FIG. 181.—Types of commercial catch-basin covers.

the adoption of a few standard types designed to meet conditions usually encountered in practice. Figure 180 shows the standard inlet casting used in Boston where the preference is for rectangular rather than circular inlets. Although ruts are more likely to form in the pavement adjacent to rectangular covers, this shape has the advantages of giving a maximum of grate opening for a given width of casting, and also of a greater stability against tipping. The corners of the support are rounded so that the grating cannot be dropped into the basin. The D-frame is not as popular as it was formerly, and where it is still used on Boston streets the corners of the support are rounded as for the square inlets. The adoption of heavier standards for both inlet and manhole castings has been made necessary by the increasing wheel loads to be supported. A preference still exists for certain types, although it is often difficult to secure from an engineer a definite reason for his liking for some of them. For example, the Concord grate, shown in

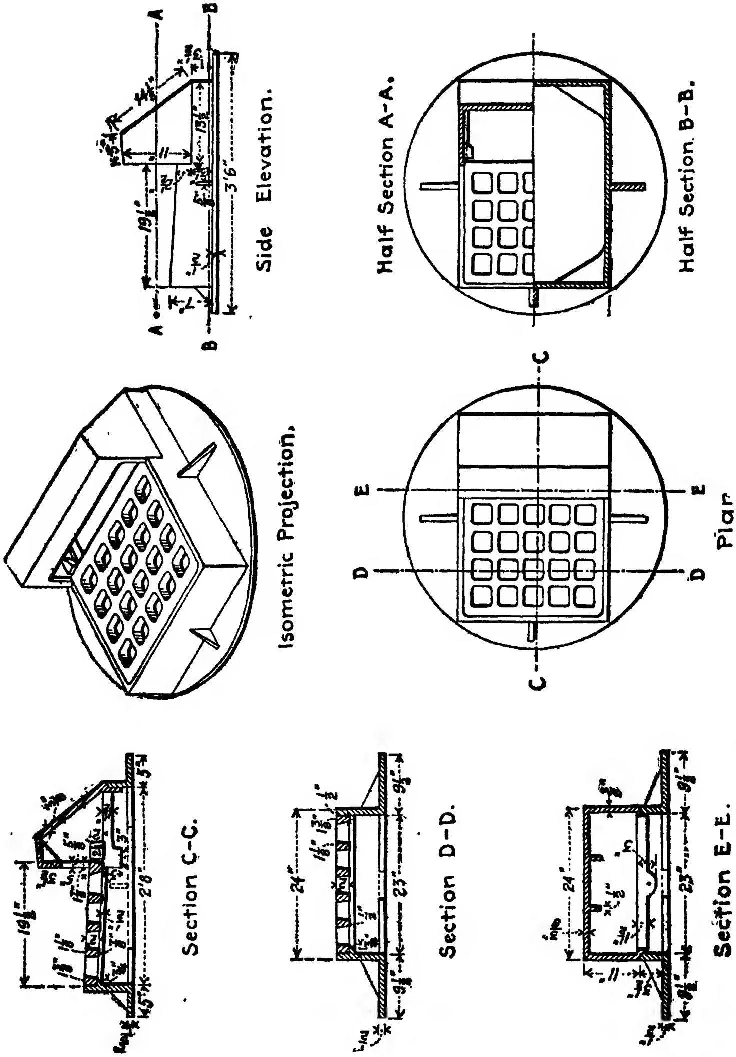


FIG. 182.—Inlet head of Borough of the Bronx, New York City.

Fig. 181, has been used for many years in New England for inlets in the gutters of streets not subject to heavy standing travel, that is to say, where the gutters are not likely to have heavy wagons drive into them to unload their contents. Of all the grates, it probably affords the easiest means for the storm water to enter the inlet, provided it does not become clogged with leaves, which experience shows have a marked tendency to accumulate. The North Berwick catchbasin is of much the same type except that the head is heavier and there is no entrance for water around the rim of the head.

The Borough of the Bronx uses a cast-iron inlet head shown in Fig. 182, which has a curb opening as well as the gutter grate. Whatever type is adopted should afford an opportunity for securely bedding the frame upon the masonry of the catchbasin or inlet, for otherwise it will become loosened speedily and in rocking under passing vehicles it will destroy the pavement about it. The North Berwick catchbasin frame is made for both 18- and 24-in. inlets, the Concord grates are made for 6- to 24-in. inlets, the Merrimac catchbasin frame is 24 in. square, measured on the cover, and the D-frame has a grate 24 in. wide and 26 in. long. In some cases the cover is in two pieces.

The material from which the frames and covers are made is rarely definitely specified. If anything is said about it, other than that it must be cast iron, the requirements are seldom more definite than that it must be of good quality and make castings strong, tough, and of even grade, soft enough to permit satisfactory drilling and cutting. It is not unusual to note a requirement that the metal shall be made without any admixture of cinder iron or other inferior metal, and shall be remelted in the cupola or air furnace. The physical test usually required is that for the metal entering into cast-iron pipe larger than 12 in. As a matter of fact, it is not likely that the metal of these castings is often tested or that the castings are inspected at the foundry. It is well to require them to meet the standard specifications for gray-iron castings of the A.S.T.M. (see Vol. II). The only additional requirements which are needed to make them apply to catchbasin castings are clauses relating to the coating of the castings and similar minor details. The coating employed is usually an asphaltum, coal-tar, or graphite paint.

MANHOLES

Although manholes are now among the most familiar features of a sewerage system, they were not used extensively until some time after many large sewers had been constructed. They were introduced to facilitate the removal of grit and silt which had collected on the inverts of sewers having a low velocity of flow. Before that time, when a sewer became so badly clogged that it had to be cleaned, it was custom-

ary to dig down to the sewer, break through its walls, remove the obstruction and then close in the sewer again, ready to cause the same trouble at a later date. The opposition to manholes seems to have been due to a fear of sewer air escaping from them, something which is not surprising in view of the contemporary accounts of the evil odors from defective drains. The engineers of the London parishes finally succeeded in obtaining authority to construct manholes, after they were able to prove that it was much cheaper to remove the grit from sewers through them than to break a hole in a sewer each time it had to be cleaned. It was not until later, however, that the value of manholes on small sewers became recognized, and the principle became established that there should be no change of grade or alignment in a sewer between points of access to it, unless the sewer were large enough to enable a man to pass through it readily. There is one modification of this rule which has been permitted to some extent, consisting of the use of a lamphole at changes in grade and more rarely at changes in alignment. Some engineers omit a manhole when it is closer than 200 ft. each way from other manholes, and substitute a lamphole. Such practice has never been general, and the use of lampholes in any situation is not regarded with favor by most engineers.

Manholes are usually placed from 200 to 500 ft. apart, but on large sewers, say where the diameter is 48 in. or larger, some engineers are using spacings of 1,200 and 1,500 ft., since inspections can be more easily made or work carried on under less trying conditions than with smaller diameters.

After the general acceptance of the principle that manholes should be placed at changes in line and grade in small sewers, there was a tendency for a time to go to the opposite extreme and put them in at too frequent intervals. This is objectionable because of the unnecessary cost and the inevitable injury to pavements caused by the presence of manhole frames in the roadway.

The great majority of manholes are constructed of brick, although under some conditions concrete may be used to advantage, particularly where a large number are to be built, so that standard forms may be utilized, or where the manholes are very deep, requiring considerable masonry. The expense of forms, the delay which their preparation frequently entails, the difficulty of placing them and of fixing the steps in the concrete, and the small quantity of concrete required, usually make it more economical to employ brick upon ordinary manhole construction.

In designing manholes, they should be arranged so as to provide easy ingress and egress and be large enough for the particular work for which they are designed. In some manholes, the clearance opposite the steps is not sufficient for a man to pass up or down without considerable

difficulty. On small sewers, there should be room to permit of the handling and jointing of the 4-ft. rods used for removing obstructions and cleaning the sewer. There should be room to handle a shovel and the bottom should afford footing for a laborer working in the manhole, but should drain to the sewer. Manholes on large sewers are sometimes arranged so that a boat can be lowered into the sewer for inspection purposes.

The manholes of small sewers are usually made about 4 ft. in diameter when of circular cross-section, or about 3 by 4 ft. when an oval cross-section is employed. The same size is usually employed for all sewers except when special conditions may require manholes of larger size, as when gaging devices must be used at the bottom of the manhole, or it is desired to have considerable storage capacity in the manhole chamber to enable this to be used to flush a long line of pipe on a flat grade. Brick manholes are usually built of 8-in. brickwork down to a depth of 12 to 20 ft., although until recently the manholes upon the Cincinnati sewers have been built of a single ring of brick, and possibly this practice has been followed in some other places. Below the depth stated, 12 in. of brickwork is used as a rule. The sides are usually carried up vertically to within about 5 ft. of the top and the upper part is corbelled in or laid in the form of a dome.

In wet and yielding material, care must be taken that the unit pressures on the foundation of the manhole and the foundation of the sewer are approximately uniform, for otherwise there is danger of a settlement of the manhole, which will break the connection with the sewer. If the pressures are not normally the same, a spread foundation may be built to reduce the unit load imposed by the bottom of the manhole. When manholes are built in sewers having a diameter approximately that of the manhole, the walls of the latter are started directly from the side walls of the sewer, as shown in Fig. 190. In the case of brick sewers a ring of brickwork surrounding the opening should be laid with joints approximately radial to the center of the manhole, so as to form a cylinder to take the thrust of the sewer arch at the point where it is cut away. As a general proposition, in fact, care should be devoted to the junction of all shafts with a sewer, for the pressure of the surrounding earth is likely to bring unexpected strains on such junctions, which cannot be calculated with any degree of accuracy. The stability of the structure can be assured by avoiding details which will give an opportunity for the backfill in settling to impose heavy loads on branches or lines of junction where it is difficult to provide extra strength without high additional cost.

Where the sewer is much larger than the diameter of the manhole, the outside of the latter is usually tangent to one side of the sewer, for otherwise it will be difficult to enter the sewer and a special ladder will be

required to reach the invert. When the sewer is very large, the whole manhole may rest on the steep side of the arch, and care must be taken to bond it with the latter carefully. This may be done by having some of the bricks in the outer ring of the sewer arch and under the position of the manhole walls project out half their length to act as headers. A horizontal tread may then be built up with these bricks as a base, and the manhole wall started from it. Occasionally, on very large sewers, the manholes are built entirely apart from the sewer proper and have a shaft leading into it, as shown in Fig. 183.

The four manhole bottoms shown in Fig. 184 illustrate somewhat different types of design. The Memphis and Seattle bottoms have flat lower surfaces while the Concord and Syracuse bottoms have lower

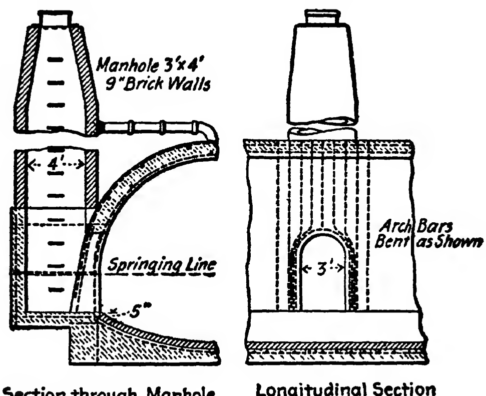


FIG. 183.—Manhole on large St. Louis sewer.

surfaces curved to correspond with the channels through them. Which type of base is best adapted for the soil at any site can only be ascertained by examination; the saving in material in the second type may be counterbalanced by an increased unit cost. While the base of each manhole illustrated was constructed of concrete, as a matter of fact a good sewer mason can lay up brickwork to form practically any channel that may be desired, and can carry the work on very expeditiously, if he is so minded.

The channels in the bottoms of the Memphis and Concord manholes are not provided with high walls, the Concord channel being nearly semicircular and the Memphis channel hardly more than that. On the contrary, the channels of the Seattle and Syracuse manholes have such high walls that they will carry all the sewage until the sewers become surcharged. It is now considered desirable to have the walls of the channel rise nearly to the crown of the sewer section, and then be

stopped in a berm, which is given a slight pitch from the wall toward the channel. The standard manhole used in Newark, N. J., for many years, which is shown in Fig. 185, illustrates this form of construction. This manhole also is so narrow that it is difficult to pass up or down the steps. The standard Philadelphia manhole bottom, Fig. 186, illustrates

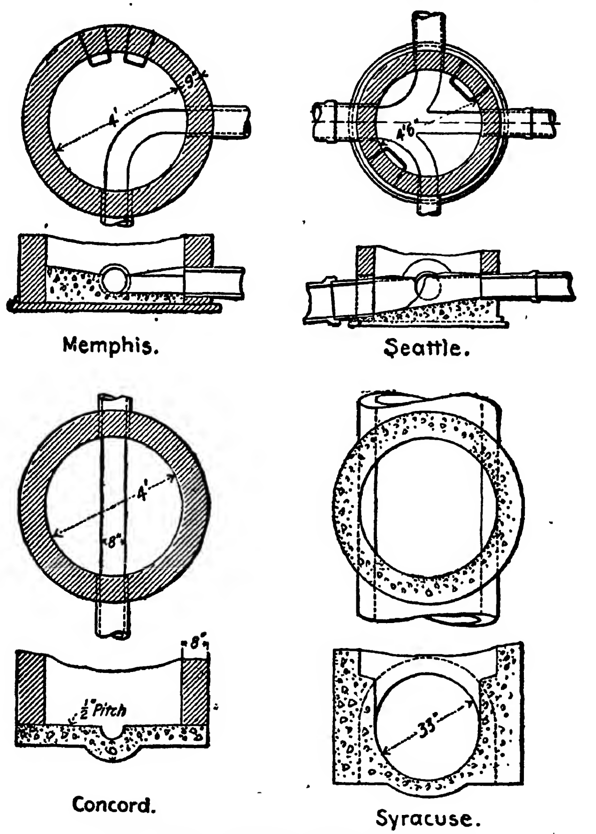


FIG. 184.—Types of manhole inverts.

the method of giving a little extra velocity to the sewage leaving the branches, by providing a steep grade for the invert within the manhole.

Changes in size or shape of cross-section of the sewer as it passes through a manhole, produce disturbances in flow with accompanying loss of head. Chamfering corners at inlet and exit, and making all changes of section by gradual transitions, assist in reducing these head losses. Carrying the sidewalls of the sewer up nearly to the crown, as above mentioned, gives greater uniformity of section at high flows with

lower head losses. There is no uniform practice in this detail, however, as the location of the berm at the top of this sidewall varies from a point mid-depth of the sewer to one level with the crown. Folwell suggests

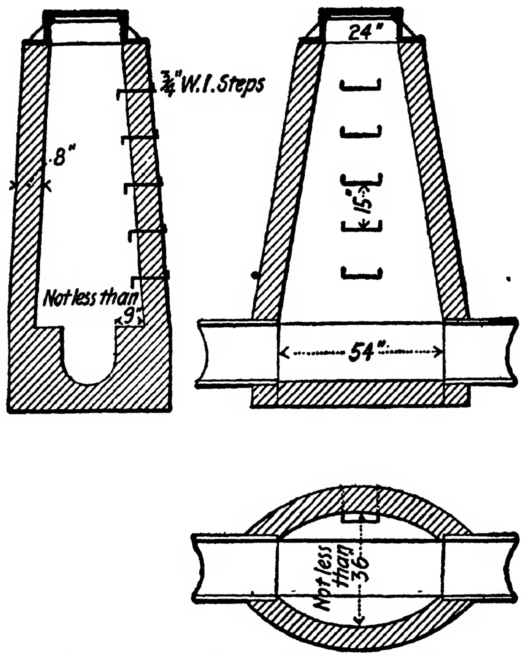


Fig. 185.—Standard manhole, Newark, N. J.

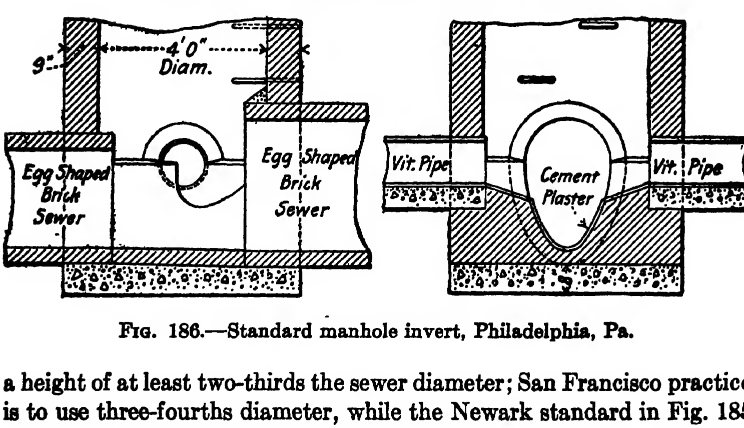


Fig. 186.—Standard manhole invert, Philadelphia, Pa.

a height of at least two-thirds the sewer diameter; San Francisco practice is to use three-fourths diameter, while the Newark standard in Fig. 185 carries the sidewall up to the crown elevation. The latter represents the more general present procedure.

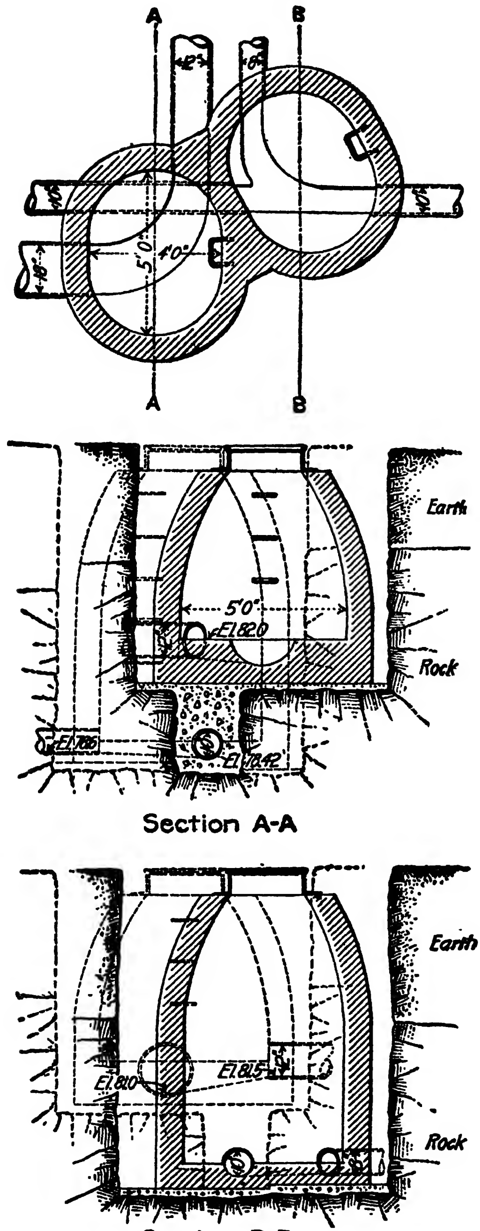


FIG. 187.—Double manhole for separate system.

Concrete manholes have been used in Syracuse on concrete intercepting sewers. Two types have been employed. In the first type the manhole has a reinforced shell 6 in. thick, running up from the sewer to within 5 ft. of the ground surface, where a funnel-shaped top begins to corbel in. The other type of concrete manhole is formed of reinforced-concrete pipe placed on end. The sections are 4 ft. in diameter and 4 ft. long, and were constructed like the reinforced-concrete sewer pipe used in the same city.

Double manholes are sometimes used where the sewers and drains are so located as to make them convenient. The structure shown in Fig. 187 was used by the authors for such a purpose on the separate sewerage and storm-drain systems of Hopedale, Mass. Each chamber is 5 by 4 ft. in plan and the dome has a depth of 4 ft. The walls are 9 in. thick.

Where underdrains are employed it is sometimes desired to afford access to them, and in such cases various expedients are employed. The most usual is to divert the underdrain a short distance to one side of the sewer, where it passes under the manhole, and to bring up a riser to the floor of the manhole. Where an underdrain is dropped along with a sewer, as in the drop manhole shown in Fig. 191, some such provision for giving access to the lower end as is there illustrated, may be provided.

The Lovejoy combination manhole, quite largely used in Boston, is shown in Fig. 188. The characteristic feature of the design is the storm drain, crossing the manhole above the sewer and provided with a large opening closed with a removable cover, which can be held so firmly in place that there will be no leakage at the joint, even when the drain is surcharged.

Drop Manholes.—The drop manhole, sometimes termed a "tumbling basin," has a mild historical interest as being the subject of patent intimidation and litigation which was an annoying feature of sewerage work in the Central States for a number of years. In 1892 a patent for the drop manhole was granted to James P. Bates, and assigned to Alexander Donahey, of Kirksville, Mo. Thereafter, whenever a city adopted plans for a sewerage system with drop manholes, it was likely to receive a notification of litigation for infringement of the Bates patent unless a license fee, usually $10 per manhole, was paid. The sum demanded was so small that the city counsel usually advised its payment, although city engineers strongly fought against it. Finally the city of Centerville, Iowa, decided to test the matter in the courts and refused to pay a license. Suit was brought, but on Apr. 16, 1907, the U. S. District Court sitting at Keokuk, ruled, before the defense had introduced its testimony, that the drop manhole had no patentable features. That ended the matter.

The drop manhole shown in Fig. 189 was constructed on Staten Island on a sewer 6 ft. 9 in. by 4 ft. 6 in. It has a 20-in. cast-iron drop imbed-

ded in concrete, for the dry-weather flow, and it will be observed that the general arrangement is such that even in times of heavy discharge the flow down this drop pipe probably serves to form a cushion at the bottom

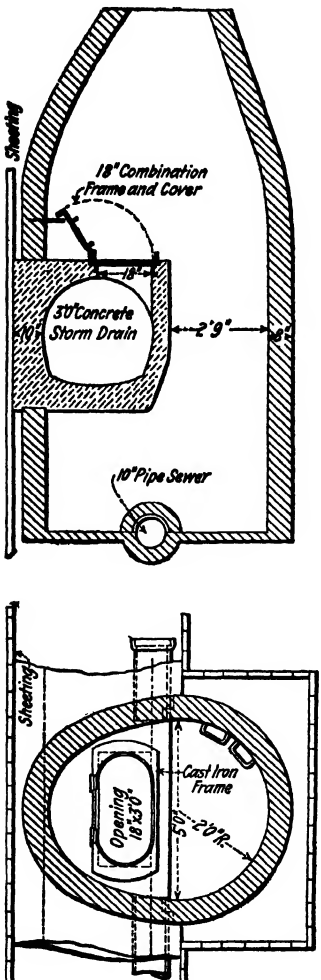


FIG. 188.—Lovejoy combination manhole (patented).

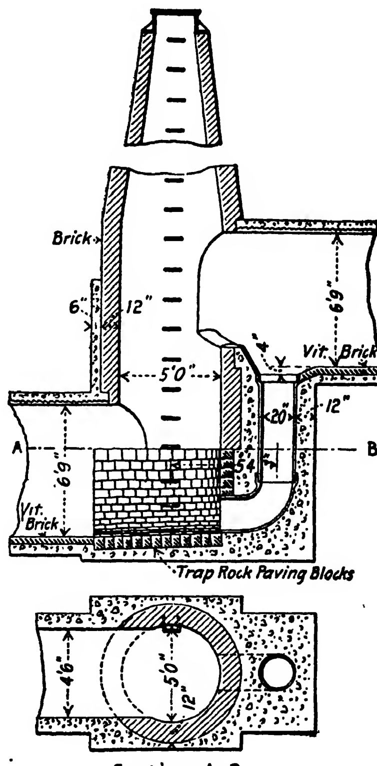


FIG. 189.—Drop manhole, Staten Island, New York.

of the manhole, to receive the bulk of the storm-water flow. It may be added as a matter of interest that on one sewer on Staten Island there are 29 drop manholes in a length of 7,883 ft. Figure 190 shows a drop

manhole built in Newark, N. J., which is rather unusual on account of its location at the head of a large oval brick sewer 4 ft. 3 in. high, into which two circular sewers discharge at different elevations. The drop

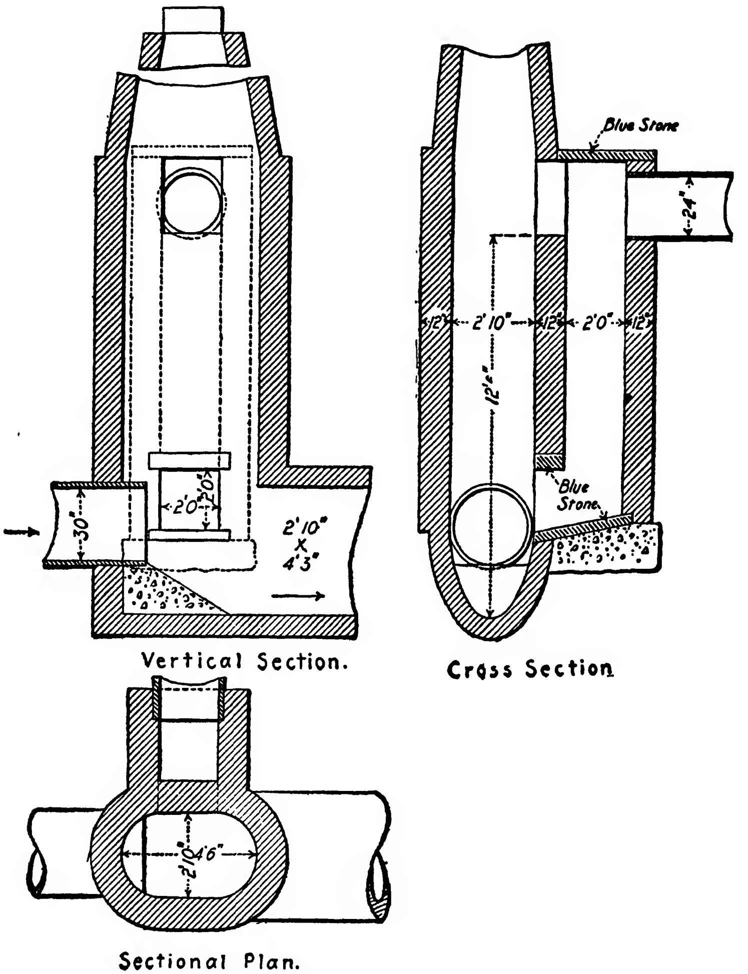


FIG. 190.—Drop manhole, Newark, N. J.

manhole shown in Fig. 191 was constructed at Medford, Mass., under the direction of T. Howard Barnes. He stated[1] that the sub-drain inspection hole had been found very convenient; it served as a well

[1] *Eng. Record*, 1897; **36**, 472.

through which to lower the adjacent ground water, when making connections with existing sewers.

Where sewers, especially small laterals, enter a manhole above the invert, there should be no shelf or flat slope upon which solids can accumulate and cause offensive odors. Such a sewer should preferably drop outside the manhole and enter at the bottom as in Fig. 191.

Wellholes.—Deep manholes in which the sewage is dropped a considerable distance from one elevation to another are sometimes called drop manholes, although that name belongs to the type just described, and they are more frequently termed "wellholes."

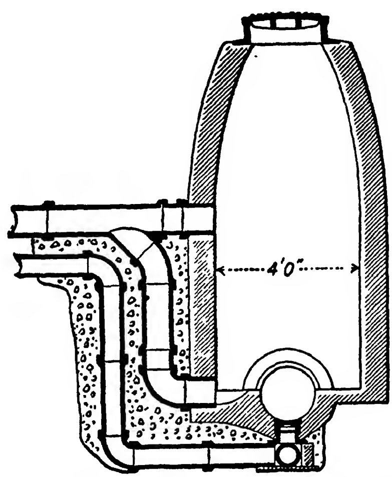


Fig. 191.—Double drop manhole, Medford, Mass.

A wellhole 65¾ ft. deep from the surface of the ground to the bottom of the invert (Fig. 192) was built in 1893 in Petrie Street, Cleveland, where the roadway was carried on a very deep fill. In order to check the velocity of fall of the sewage the latter dropped at intervals of 5 ft. on stone flagging, having a thickness equal to that of two courses of brick, placed as shown in the illustration. The connection from the bottom of the manhole to the 6-ft. culvert, was of a flexible character, as indicated in the sketch, owing to the probability that there would be some settlement under the fill in the course of a few years. After this settlement had occurred it was proposed to calk the joints of the connection thoroughly from the inside. Whether this was done cannot be learned but the structure served its purpose satisfactorily for about 10 years, when it was abandoned on account of the reconstruction of the Petrie Street sewer.

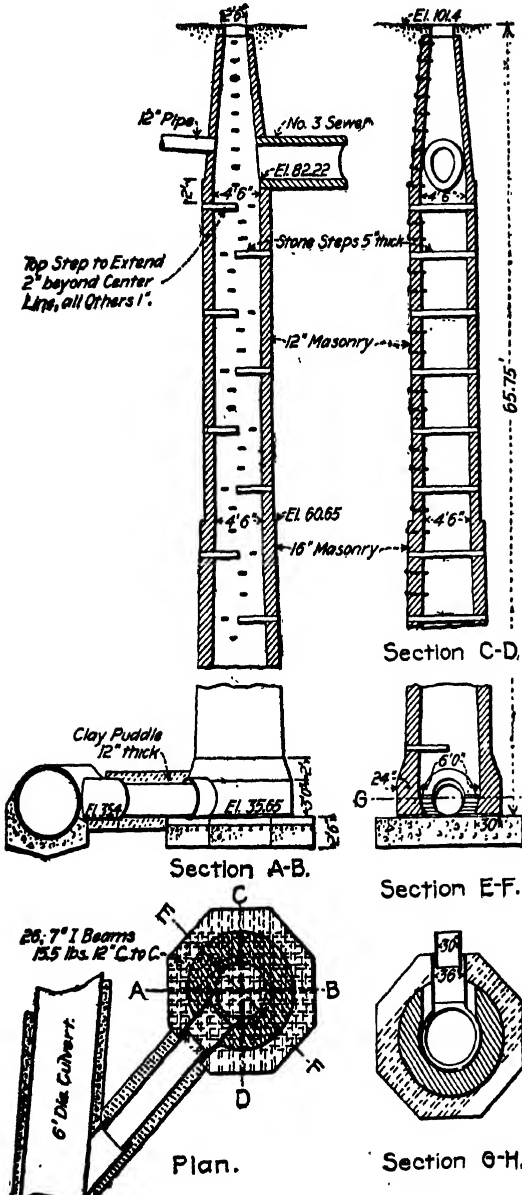


FIG. 192.—Wellhole, Petrie Street sewer, Cleveland, Ohio.

Some very deep wellholes have been constructed at Minneapolis, in connection with the sewers built to discharge storm water into the Mississippi. The greater portion of the city served by these sewers is from 80 to 100 ft. above the river. Along the river bank is a drive and park which made it necessary to build the wellholes some distance from the river. For example, the typical wellhole shown in Fig. 193[1]

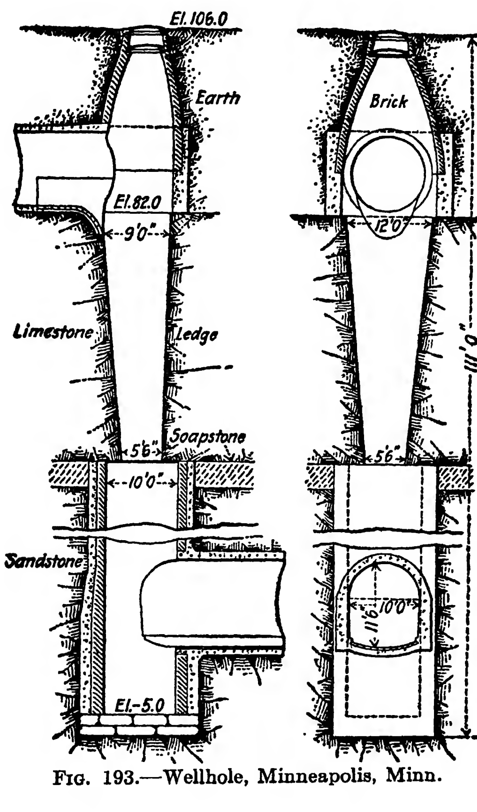


FIG. 193.—Wellhole, Minneapolis, Minn.

is 340 ft. from the outlet. Where the drop is through hard limestone the section is not lined but given a funnel shape, which is advantageous in concentrating the sewage in the center of the lined portion of the wellhole. This latter has a lining of granite block in a backing of concrete, and the outlet sewer from it starts at an elevation which gives a deep sump in the bottom of the wellhole, forming a water cushion to prevent erosion of the lining by the falling waters.

[1] *Eng. Record*, 1911; **63**, 382

On some of the tunnel sewers in the Borough of Brooklyn there are manholes from 65 to 83 ft. in depth into which sewers discharge at distances of 25 to 40 ft. above the invert of the main sewer below. Below these shaft manholes the invert is paved with granite blocks laid in portland cement for a distance of as much as 30 ft. Furthermore, although the trunk sewer is in a tunnel at this place, an extra heavy bottom is constructed below the shaft and manhole for a length of about 14 ft.

The use of drop manholes and other special details to give a sudden drop in grade is not regarded with favor by some designers. For example, W. W. Horner, of the St. Louis Sewer Department, stated[1] that

> . . . the tumbling basin introduces unknown factors into a sewer system, which we now think best to avoid, if possible. It is questionable whether the basin really acts to advantage under extreme conditions. Such construction is very expensive, for if the sewer is deep enough above the basin, it is too deep below, involving excessive excavation; also, if it is supposed to check the velocity, much larger sewers are required for the flat grade. The present practice is to design the sewers carefully at all points and to take advantage of all the natural fall, in order to decrease the size of the sewers; then to build them strong enough to take care of the resulting high velocities.

Where sewers are built in deep rock cut, the high cost of excavation frequently has led in St. Louis to the adoption of a rectangular cross-section for the sewer. By making the sewer narrow and high the amount of excavation will be materially decreased, but as the ratio of the height to the width increases, the section becomes less efficient from the hydraulic viewpoint, requiring a greater wetted area for the same capacity. A number of conditions must be fulfilled in such cases, and the best section can only be determined by a number of trial calculations.

Flight Sewers.—A considerable fall must sometimes be provided in a sewer, and while a drop manhole or wellhole always affords a means of changing grade sharply, the lower sewer which leads from such a shaft may be so deep that any prolongation of it should be avoided if a less expensive structure can be made to serve. The flight sewer, which gets its name from its resemblance to a flight of stairs, is occasionally used in such situations. It has a steep grade, but steps in the invert tend to check the velocity of the current; the resistance they offer probably diminishes with the depth of the sewage, and if the descent is long, care should be exercised to ensure massive, durable construction, and freedom from obstruction to flow at the bottom of the flight, which may be seriously strained if the sewer should ever run full. Two examples of

[1] *Eng. News*, 1912; **68**, 426.

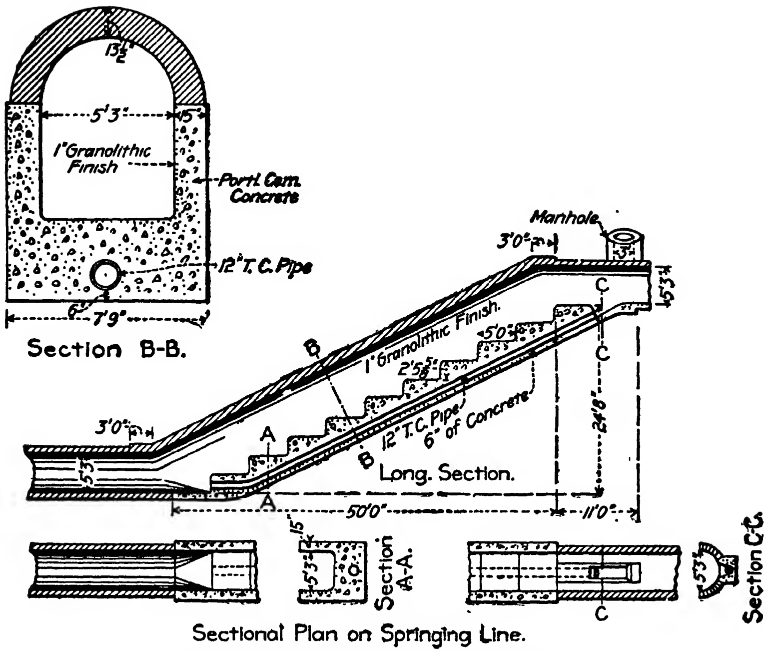


FIG. 194.—Flight sewer, Philadelphia, Pa.

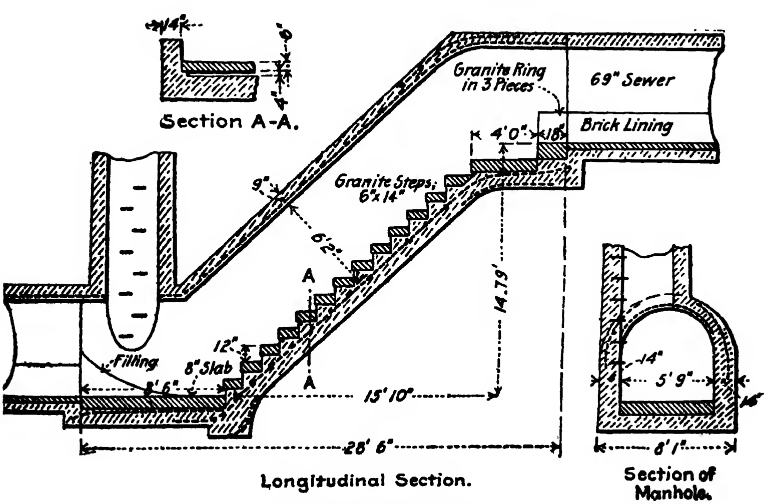


FIG. 195.—Flight sewer, Baltimore, Md.

such a sewer are shown in Figs. 194 and 195, from *Engineering Record;* the first [1] as a small circular channel within the concrete base to carry the dry-weather flow while the second has no such provision.

The flight sewer shown in Fig. 194 is a part of the Indian Run sewer in Philadelphia. The total length of this special section is 61 ft., and in that distance there is a drop of 24 ft. 8 in. The granolithic finish of this section was a mixture of one part cement, one part sand, and one part granolithic grit. On the risers this mixture was placed against the face of the forms in advance of the bulk of the concrete filling, with a minimum thickness of 1 in. After the forms were removed the face was at

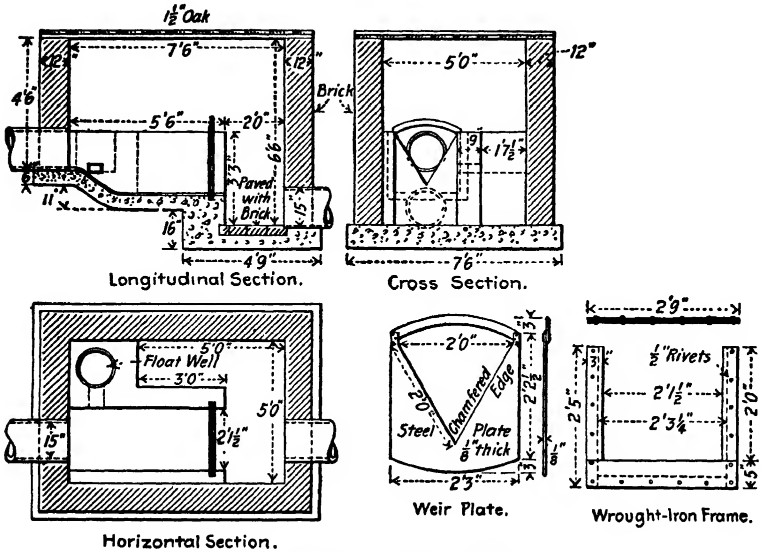


Fig. 196.—Gaging manhole, Liberty, N. Y.

once brushed with a thin plaster of equal parts of sand and portland cement.

Special Manholes.—The gaging manhole shown in Fig. 196 was built at Liberty, N. Y., from the plans of Wise & Watson, of Passaic, in 1900. This manhole is provided with a triangular weir. For a discussion of the capacities of triangular weirs, see Chap. IV.

Information regarding other forms of gaging manholes is given in Chap. X, on gaging storm-water flow in sewers.

A manhole for an unusual purpose is illustrated in Fig. 197,[1] and it also is of interest in that it is one of the very few structures where life has been lost owing to the harmful effect of sewer air. This structure is

[1] *Eng. Record*, 1909; **60**, 252.

at the end of the Los Angeles sewer outfall where it discharges into a wood-stave pipe that carries the sewage 900 ft. out to sea. The old sewer outfall was badly disintegrated in places by the sewer air, where it did not run full, and this gate chamber was designed to keep the lower portion of the conduit under a slight head. It has a gate across the main

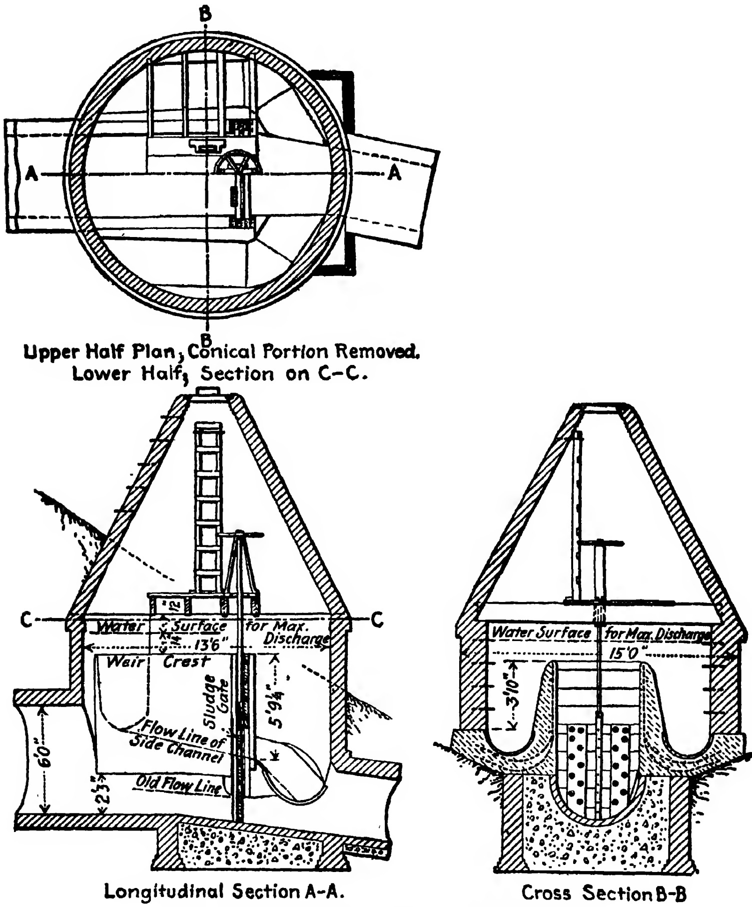


FIG. 197.—Gate manhole, Los Angeles outfall.

central channel running through it, and on each side of this channel is a dam or weir. By closing the gate the sewage is forced to rise and find an outlet over the crests of the two weirs. One of the engineers of the city lost his life in 1909 in manipulating the hand wheel by which the gate was raised and lowered. With a companion he moved the gate a

number of times, and the companion reported that whenever the gate was near its seat the violent rush of sewage below the bottom of the gate gave off gases which caused extreme dizziness. They were several times forced to come to the surface and lie down; the engineer lost his life on one of these occasions. Instead of leaving the manhole he stood partly out of it, his arms resting on the manhole frame and his feet on the ladder. Suddenly he was seen to drop, and when his companion hurried to the gate chamber his body could be seen resting on one of the steep side inverts, from which it slipped into the wood outfall; a few days later it was found floating in the water, near the outlet.

Manhole Steps.—In shallow manholes, steps are sometimes formed by leaving projecting bricks at the proper points, about 15 in. apart

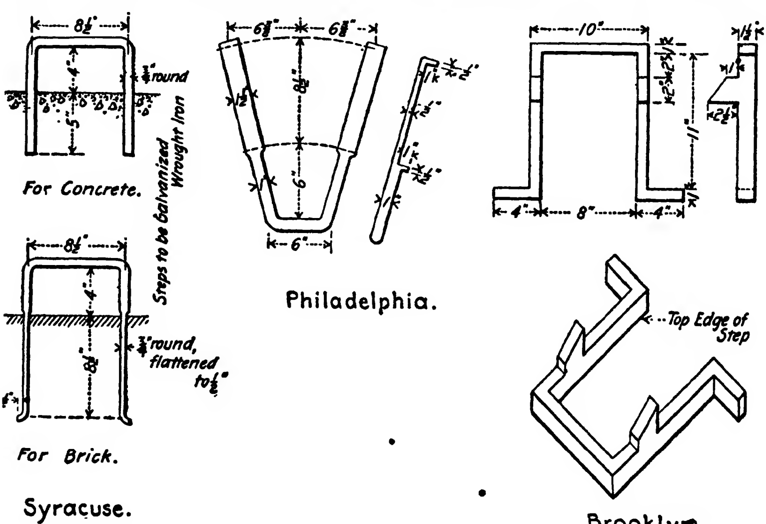


Fig. 198.—Types of step forgings for manholes.

vertically. This is an old practice and while not approved by many engineers, was at one time commonly used. Such steps are objectionable because they are sometimes slippery, when it is difficult to use them safely, and, moreover, they are easily broken.

A common method of providing steps at the present time is to construct them of forgings, which are bedded in the brickwork or concrete. Three types of these steps are shown in Fig. 198. Sometimes steps are formed by straight rods inserted in the masonry in such a way as to form chords of the brickwork ring, with the center of the step at least 4 in. from the brickwork. The steps are usually placed from 12 to 18 in. apart vertically and somewhat staggered; a number of cities seem to be

in favor of a vertical spacing of 15 in. The authors have found the step of the type marked "Syracuse" in Fig. 198, satisfactory, but experience with it indicates that the blacksmiths who forge the steps must be cautioned to follow the dimensions accurately, for otherwise there will be trouble in fitting the steps into the joints of the brickwork. Fig. 199 shows a cast-iron step used in Boston where the shaft must be kept free from any projections from the wall.

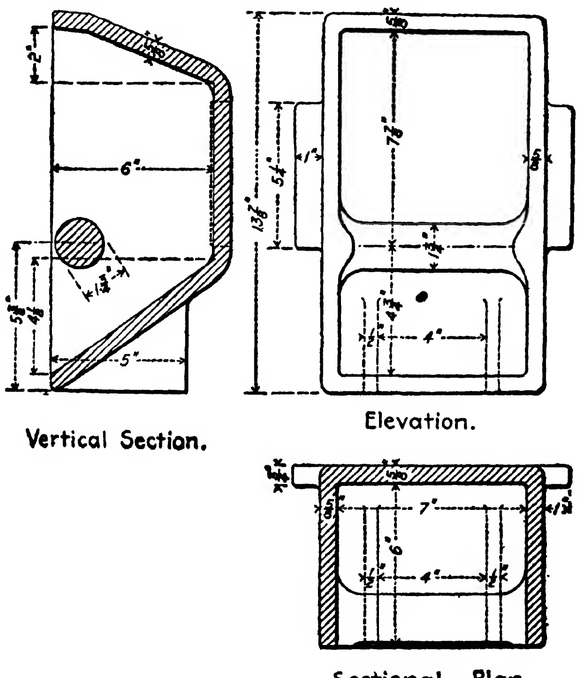


FIG. 199.—Cast-iron box step, Boston, Mass.

On some work that has come to the authors' attention, it has been found that the material used for making forgings for steps has rusted to such an extent that the steps are practically useless. In one instance this condition was found where genuine wrought-iron steps had been specified. The authors' present practice (1928) is to use cast-iron steps as shown in Fig. 200.

Manhole Frames and Covers.—It is only within the last few years that the design of manhole frames and covers has been given serious attention. The selection of shape, size, or pattern for these castings has been based largely upon personal preference which varied considerably in different localities. This necessitated the carrying of many

patterns by the manufacturer, one maker having as many as 2,000 different patterns, sizes, and shapes in his shop. This condition of affairs prompted the Industrial Association of San Francisco to request the Division of Simplified Practice of the Department of Commerce to call a conference of interested parties to consider the problem of agreeing upon a few standard designs adequate to meet the range of requirements usually encountered. At this conference (1924) the objects to be

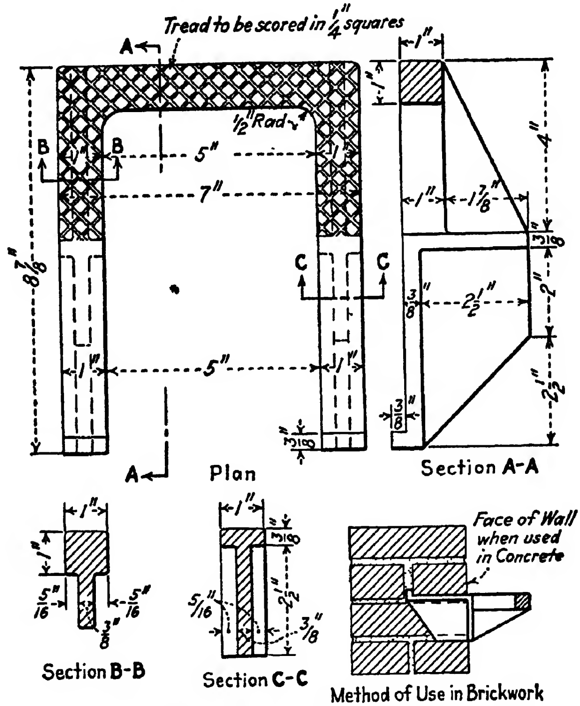


Fig. 200.—Cast-iron manhole step.

attained in the design of castings (particularly for manholes) were enumerated as follows: Safety, so that covers will not slip off; convenience of repair and replacement, necessitated by the wear from traffic; strength sufficient to stand up under increasing wheel loads; freedom from rattle and noise; cheapness; possibility of adjustment with the wearing down of pavements to prevent uneveness with its accompanying inconvenience to traffic and increased wear on the pavement; sightliness; ventilation to remove accumulating gases so as to increase safety of workmen in manholes and sewers; protection again ingress of water, especially for manholes used by telephone or electric light com-

paines; protection against ingress of lighted cigars and cigarettes; and protection by locking devices against removal for dumping of refuse into the opening.

It might be added that the cover should be flat and lie in the plane of the pavement so that it will not interfere with traffic nor cause excessive wear of the pavement. Covers should be interchangeable for convenience in replacing those lost from theft or breakage. In the Borough of Manhattan, with 30,000 sewer manholes, about 1½ per cent of such replacements are required each year. The cover should be corrugated or provided with bosses to prevent slipping. Circular tops are in almost universal use for sewer manholes. They are inherently stronger than rectangular ones, and have the advantage that it is impossible to drop the cover into the manhole.

Frames are usually from 6 to 12 in. in height, depending partly upon the type of pavement in which they are to be installed. Practice as to clear opening varies widely; a 24-in. cover, allowing 22-in. clear opening, is generally satisfactory. It is more convenient to enter through a larger opening, but the cost and the likelihood of breakage increase materially with the size; and comparatively little reduction in opening is practicable. In general, frames weigh from 250 to 500 lb., and covers from 100 to 150 lb.

Manhole frames and covers are usually of cast iron, but semisteel or cast steel may be used. T. J. Corwin[1] describes tests of large manhole covers (28 to 36 in.) made for the Pacific Gas and Electric Company, which indicate that semisteel is the most advantageous material for such large covers. Several designs of cover were tested.

Wear on manhole frames in large cities results in very considerable expense for renewals, the cost of labor and of cutting and replacing pavement around the frame generally being of most consequence. To reduce this cost to a minimum, the Chicago Sewer Department has designed a reversible manhole top consisting of three pieces, a base, curb, and lid, as shown in Fig. 201. The base is permanent and remains in place during repairs. The curb is a reversible ring with vertical sides so that it can be removed without damage to the surrounding pavement when the top has worn sufficiently to require reversal. When the reverse edge is also worn down, the curb can be replaced for about $6 plus $5.50 for labor cost. The frame thus has a longer life and its replacement is a simple and inexpensive operation causing only slight interruption of traffic.

Due to the large increase in motor vehicle traffic, the old covers, of which the sewer and water departments have about 300,000, are wearing out rapidly. Few covers in the downtown district last more than 15 years and it is now the practice to install new ones when pavements are

[1] *Trans. Am. Soc. C. E.*, 1927; **91**, 991.

relaid. The life of pavements in this district averages 7 years. The lid is designed with sufficient strength to carry the heavy wheel loads. Tests made by dropping a 1,200-lb. weight on the curbs and lids from different heights, showed the reversible type to have greater resistance to impact than the standard types.[1]

The Boston standard manhole frame and cover in Fig. 202 may serve to illustrate the usual practice at the present time (1928) in these castings. The cover is heavier than the older standards and the deeper ribs on the lower side make it more difficult for boys to remove since it must be lifted clear of these. It is a perforated cover, but the number of holes

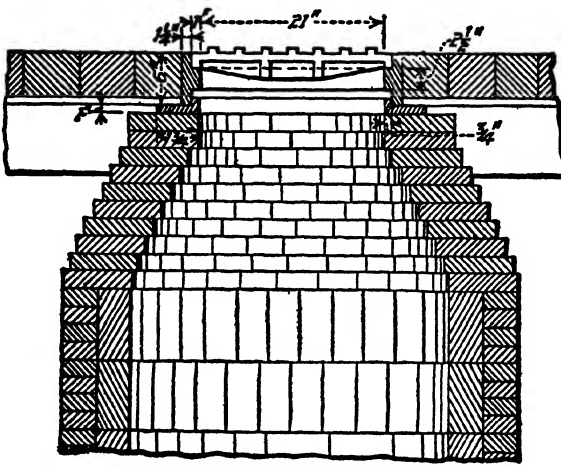


Fig. 201.—Reversible manhole frame, Chicago.

is not large. The clear opening in the frame is practically 24 in. in diameter.

Within the last 40 years it has been rather common practice to use perforated manhole covers to provide for ventilation. In some designs, it appears as though the attempt had been made to provide as many holes as possible. Sometimes the perforations were in the bosses, with the object of excluding water. In general, the holes were larger at the bottom than at the top, to avoid plugging with sticks and dirt. It will be noted that the Boston manhole cover has perforations 1 in. diameter at the top and 1¼ in. at the bottom. A recently designed manhole cover for Manhattan Borough[2] has six 1-in. holes inclined 45 deg. from the vertical and so placed that the lower end adjoins a reinforcing rib; this is intended to prevent sticks being poked through the holes, unless in so short pieces that they cannot obstruct the sewers. These covers were

[1] *Eng. News-Record*, 1922; **89**, 928.

[2] *Eng. News-Record*, 1924; **92**, 774.

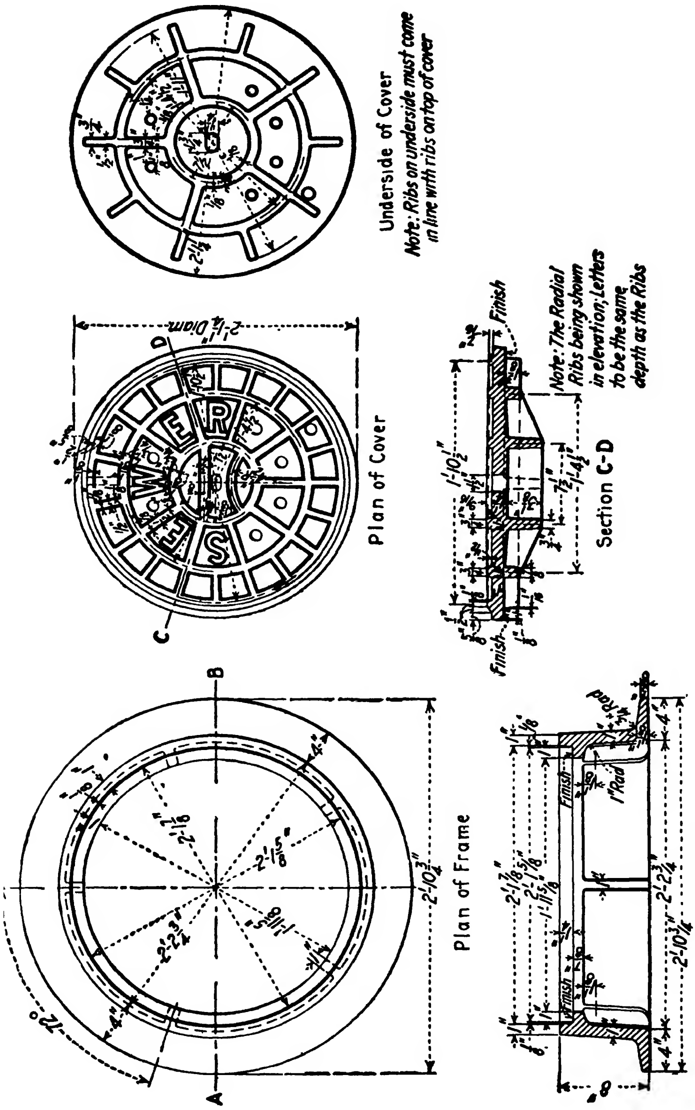


FIG. 202.—Standard manhole frame and cover, Boston, Mass.

designed for a live load of 800 lb. per linear inch of tire tread, with a maximum concentration of 13,000 lb.

Just how much storm water enters through the perforations in covers is difficult to estimate, because of the leakage into the sewers from other sources. Just before the joint outlet sewer in northeastern New Jersey was completed, there was a very heavy storm. There were 125 miles

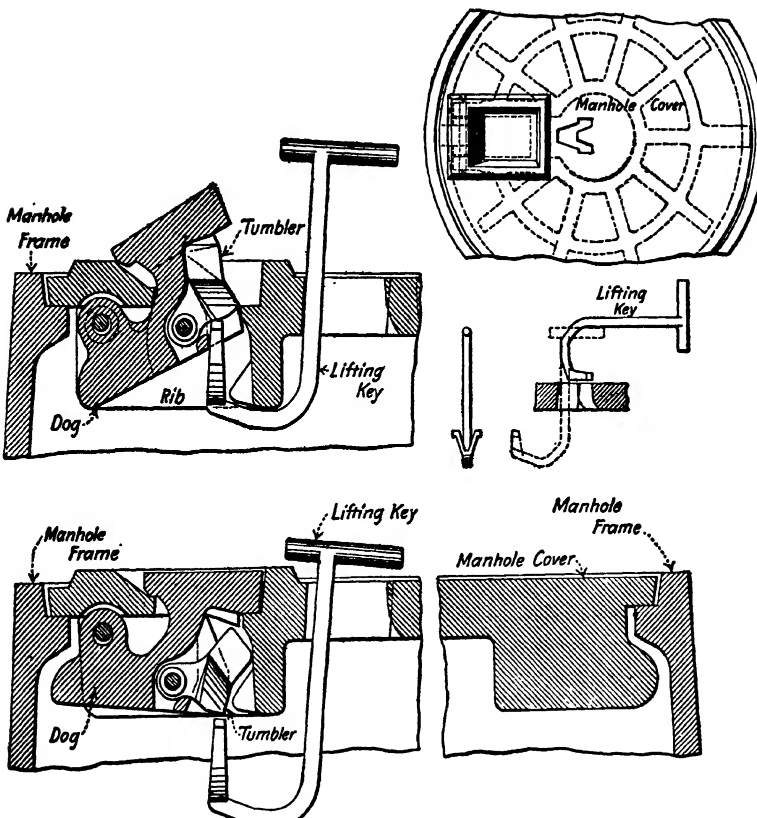


FIG. 203.—Locking device for manhole or catchbasin cover, Boston, Mass.

of sewer in the system at that time, and about 1,965 manholes having perforated covers. No catchbasins were connected and no roof water was supposed to be admitted. According to the chief engineer, Alexander Potter, as nearly as could be ascertained 3,000,000 gal. of water entered the system in 24 hours through these covers, or an average of 1.1 gal. per minute per manhole.

So much storm water has found its way into separate sewers in some places that there has been a recent tendency to use tight lids or those having few and small openings in an effort to prevent or reduce leakage through manhole covers.

The locking device for manhole and catchbasin covers shown in Fig. 203 was designed and patented by R. J. McNulty, mechanical engineer of the Sewer Service of the Boston Public Works Department. The lock is a cast-iron dog hung loosely on a pin, with a lug projecting rearward to engage the underside of the ledge of the manhole frame. The dog also has an arm projecting forward to form a closure for the

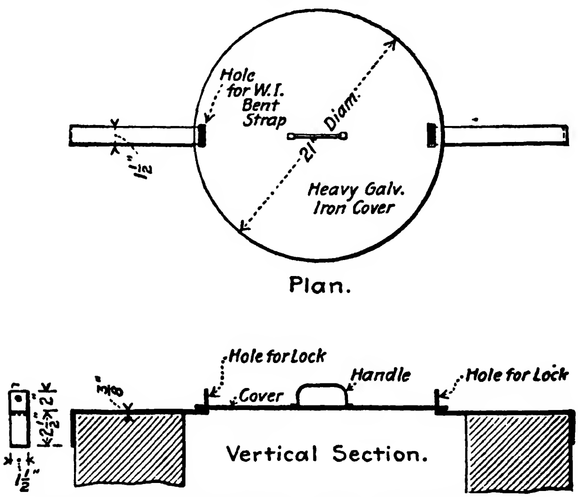


FIG. 204.—Locked manhole cover, Philadelphia, Pa.

opening in the cover. Beneath this arm are three ribs, between which are pivoted two tumblers so shaped that gravity causes them to lie normally under the ledge of the cover. It is then impossible to unlock the cover because the tumblers engage the ledge on it and prevent the lug on the dog from disengaging the ledge on the frame. The unlocking can be accomplished by inserting the two-pronged lifting key through the special hole in the cover, turning it 180 deg. and then lifting it, which will lift the tumblers and dog, allowing the cover to be removed. When the cover is replaced, the dog and tumblers fall by gravity into position and lock the cover automatically. The cover cannot be unlocked with a bent wire, like many locking covers. It is made by the McNulty Engineering Company of Boston. Present practice in Boston is toward making relatively deep ribs on the under side of the lighter covers so that they must be lifted

clear in order to be removed. This is said to be a better guarantee against removal by boys than locking devices. Many of this type have been used in Boston to prevent the dumping of ashes and other refuse into manholes and where the displacement of a cover would be particularly dangerous to traffic.

It is sometimes necessary to lock the entrance to a manhole more certainly than can be accomplished by any of the catches which are used to some extent to prevent the removal of the covers. A cover for

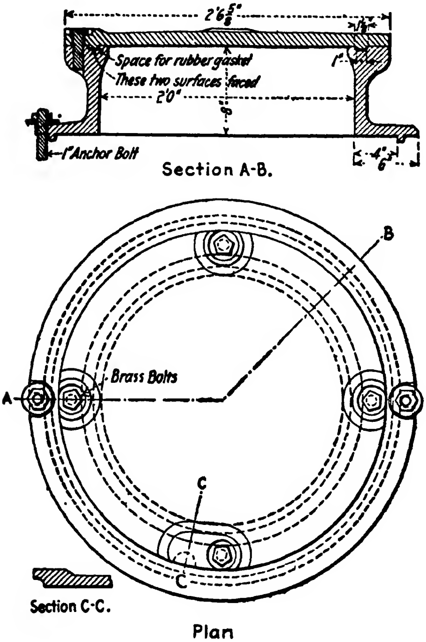


Fig. 205.—Watertight manhole frame and cover.

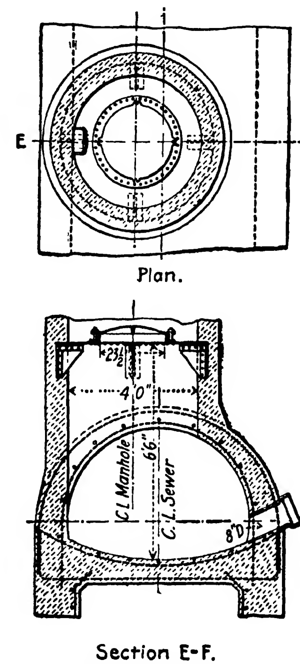


Fig. 206.—Watertight manhole diaphragm.

the purpose is shown in Fig. 204. It is used at the top of a wellhole, in Philadelphia, in which a gaging machine is kept, and immediately on top of it rests a manhole frame and tight cover. The locked cover is a circular plate 21 in. in diameter, and is supported by two flat wrought-iron bars ⅜ in. thick, which are bent at each end to fit into a hole cut for the purpose in the cover. There is a ½-in. hole for a Yale padlock in the end of each bar, and two padlocks are used to fasten the cover.

A watertight manhole frame and cover, designed for the sewerage system of Concord, Mass., by one of the authors, is shown in Fig. 205.

No detailed explanation of this design is necessary, except that the brass bolts were equally spaced so that the cover will fit in any of four possible positions. Figure 206 shows how a manhole frame and cover of this type were used inside a manhole in Louisville, where it was necessary to provide against flooding neighboring lands with water from the sewer, when the river into which it discharges is at a high stage.

Lampholes.—It has been intimated already in several places that the authors have not found occasion to use lampholes on their work. In cases where they might have been employed, it was considered that the additional cost of a manhole was well warranted by the advantage of accessibility to the sewer which it presents. It is true that by means of mirrors attached at proper angles to a rod lowered into a lamphole, with a good light reflected down the lamphole by the mirrors, it is possible to see something of the condition of the sewers in its vicinity. The main use of these shafts, however, is to enable a man to lower a light of some sort down into the sewer, so that an observer stationed at a manhole on either side of the shaft can inspect the interior of the pipe. It is frequently stated that a lamphole can be used for flushing, if a hose connected with a nearby hydrant is carefully lowered down it; this is done in some of the smaller cities having limited funds for sewer improvements, as at Paris, Tex., where there are very few manholes and the lampholes are frequently used for this purpose. Serious obstructions can seldom be removed or dislodged in this way and rodding is, of course, impossible. In the opinion of the authors, the best views in this and other countries regarding these lampholes have been well summarized by Frühling in his "Entwässerung der Städte," in the following words:

> In order to economize in manholes, these oftentimes alternate with lampholes, which are cheaper to construct and suffice to enable the flow of the sewage to be observed. This can be done either by looking down the shaft after removing the cover, or a lamp can be lowered down the shaft and can be observed from the nearest manhole, either directly or with the aid of a mirror. In most cases the character of the flow will afford information whether everything is as it should be or a clogging has arisen, and whether the cause of the latter is above or below the lamphole. The obstructions are removed from the nearest manhole, for the lamphole permits only a very slight means of ingress, such as the introduction of a hose. As far as the diameter of the lamphole is concerned, from 6 to 10 in. is enough, according to the depth of the sewer. The shaft consists of vitrified clay, concrete, or iron pipe, and more rarely masonry. The frame and cover at the top are to be placed in the roadway so that the weight coming upon them does not bear on the shaft, which would transfer it to the pipe sewer. If the lamphole is at a place where a flat grade changes into a steeper one, the cover should have ventilating holes.

In addition to what is stated in this quotation, it is desirable to lay emphasis on the necessity of avoiding any weight on the shaft. Experience shows that even the weight of the riser pipe forming the shaft will sometimes break the sewer pipe from which it rises. The disastrous experience of this sort at Memphis, mentioned in the Introduction, has been duplicated at many other places. Consequently the frame and cover, which are made like small manhole castings, should be carried by a ring of concrete or masonry surrounding but not touching the vertical pipe. Even with such precautions a lamphole is bound to be a source of structural weakness, and its use should be avoided if possible.

CHAPTER XVI

JUNCTIONS, SIPHONS, BRIDGES AND FLUSHING DEVICES

JUNCTIONS

The earliest discussion of the importance of easy curvature and of carefully guiding together the streams of sewage at a junction, which the authors have found, appears in the report of the British General Board of Health of 1852, where Roe, best known for his table of the areas drained by circular sewers of different diameters, made this statement:

> Every junction, whether of a sewer or drain, should enter by a curve of sufficient radius; all turns in the sewers should form true curves, and as, even in these, there will be more friction than in the straight line, a small addition should at curved points be made to the inclination of the sewer. I may mention a case or two in illustration. In 1844, a great quantity of rain fell in a short space of time, overcharging a first-size sewer and flooding much property. On examination, it was found that the turns in the sewers were nearly at right angles, and also that all the collateral sewers and drains came in at right angles. The facts and suggested remedy were reported to the Holborn and Finsbury Commissioners, and directions given by them to carry out the work. The curves and junctions were formed in curves of 30-ft. radius, and curves with cast-iron mouths were put to the gully chutes and drains; the result was that although in 1846 a greater quantity of rain fell in the same space of time than in 1844, no flooding occurred, and since then the area draining to this sewer has been very much extended without inconvenience. In another case, flooding was found to proceed from a turn at right angles in a main line of sewers. This was remedied by a curve of 60-ft. radius, when it was found that the velocity of current was increased from 122 (as it was in the angle part) to 208 (in the curved part) per minute, with the same depth of water.

With small sewers which it is impracticable for a man to enter, the changes in direction as well as grade should be made in manholes, as already explained, or at lampholes. If this is not done, there is a risk of a stoppage occurring at some point where its location cannot accurately be determined, and if such a thing occurs the only remedy is to dig down through the street to the sewer. By the time the obstacle has been removed, the sewer repaired and the trench filled, the desirability of avoiding such occurrences in the future will be entirely clear.

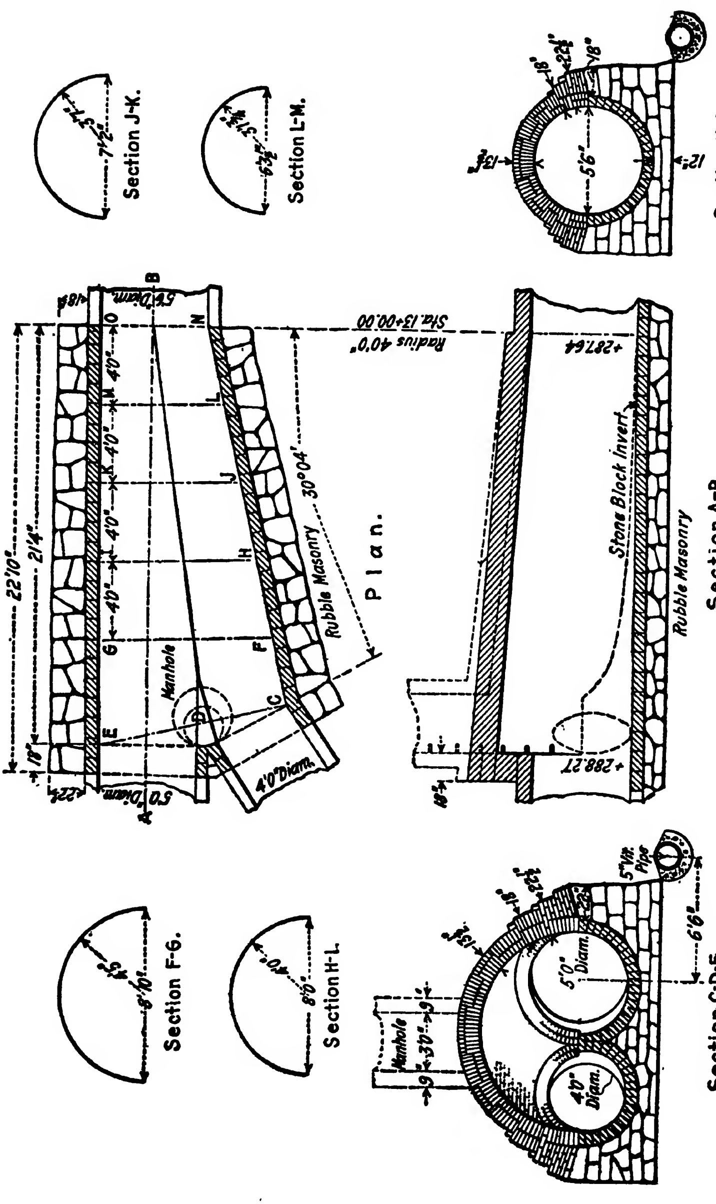


FIG. 207.—Bellmouth junction, Philadelphia, Pa.

Where the sewers are large enough to be entered, so that their junctions do not need to be made in manholes, and they come together with a horizontal angle between their axes less than about 30 deg., a special structure called a junction is required. For many years these junctions were usually of the type shown in Fig. 207, and were called "bellmouths" or "trumpet arches." The two sewers are constructed as independent channels until the outside lines of their masonry come together at the springing line of the arches. If they were continued beyond this point as independent arches, the tongue forming the support for both arches would gradually become thinner and thinner, and the roof of the junction would consequently be in danger of falling through lack of supporting strength at this point. Eventually the tongue would become so thin that even the most reckless builders would not try to carry the roof upon it. Accordingly where, at the springing lines, the outside of the arches come together, no further attempt is made to have the upper portion of the confluent sewers independent, but a large arch is thrown across the two. At the highest point of this arch, just in front of the brick wall which closes the large end of the structure, a manhole or ventilating shaft of some sort is frequently erected. Care should be given to forming the curves of the invert to the correct lines, because at these junctions there is frequently some sedimentation, due to backwater, and the inverts should offer no obstruction to the washing away of these deposits by the first storm that arises.

The structure shown in Fig. 207 was built of brick and stone, but now bellmouths frequently are constructed of concrete. Where brick is employed and the masons are experienced men, the construction of one of these junctions, even when more complicated than that illustrated, is not a difficult task; while the centers must be strong, they do not require the careful finish of a form for concrete, such as the Louisville structure shown in Vol. II. In any case, however, the principal expense for one of these bellmouths is the item for skilled labor, either for laying the brick or for making the forms. To avoid, so far as possible, any further increase in these items, some engineers have recently turned to flat-topped junctions.

A flat-topped junction constructed in Pittsburgh is shown in Fig. 208. It is a structure which was more expensive to build than most of the same general type, because it was inserted on an existing brick sewer of large size, which it was desirable to disturb as little as possible. This was rendered more easy from the fact that the sewers come together at an angle of about 45 deg., which renders unnecessary a long tongue at the junction of the invert. The roof, in this case, is a reinforced-concrete slab, and the manner in which the old brickwork has been surrounded with concrete, so as to utilize it as fully as possible, deserves attention.

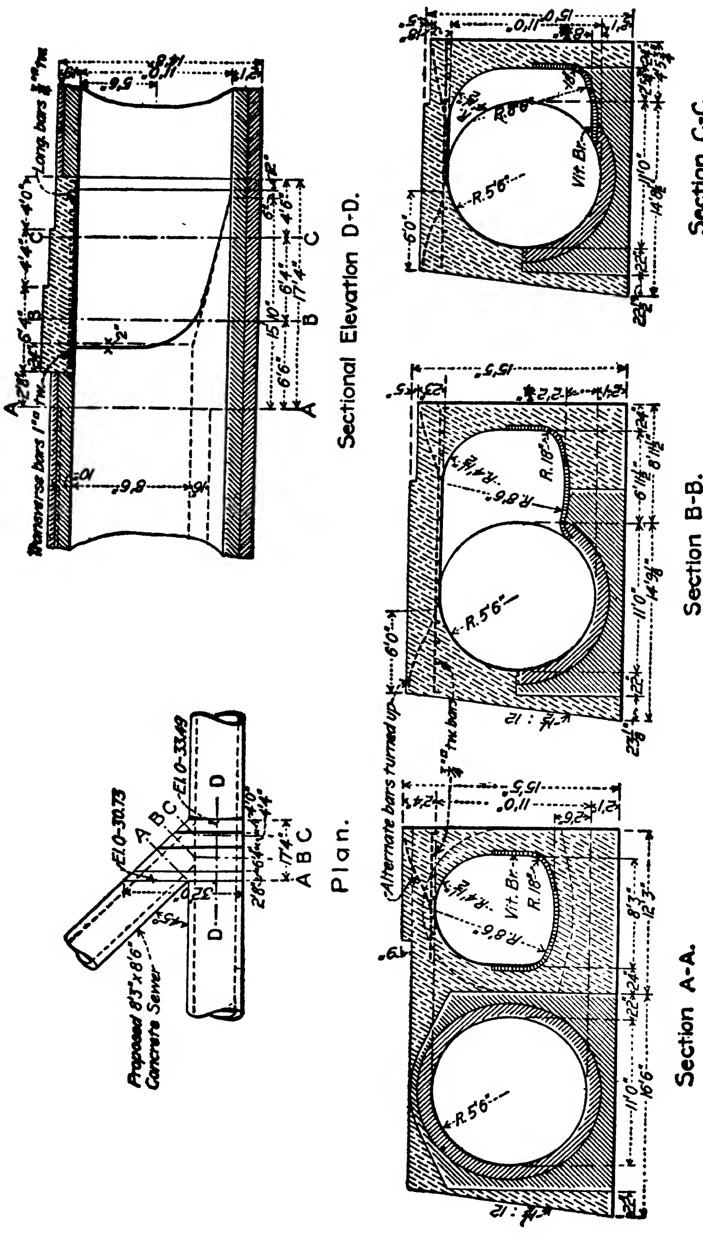


FIG. 208.—Flat-top junction, Pittsburgh, Pa.

The sharp pitch given to the new sewer where it joins the large existing sewer also deserves attention, and this feature of design will be referred to a little later.

Where flat-topped sewers are necessary at junctions, and the angle which the axes of the two confluent sewers makes is small, it is customary for the roof to be carried by I-beams. The construction of a junction of this sort in Philadelphia may be mentioned as an illustration of the general arrangement. There were two brick sewers, 10 and 11 ft. in diameter, respectively, which came together with inverts at the same elevation. Both were of the brick and rubble type used so extensively in that city. The total length of the interior of the junction structure was 45 ft. The cross-sections of the invert were worked out in the

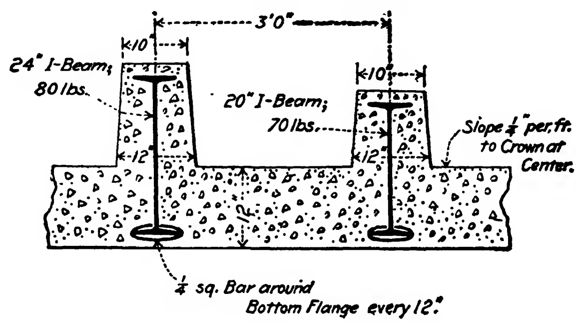


FIG. 209.—Roof detail, Philadelphia junction.

usual manner. Starting at the springing line, the walls of the junction were run up straight and given a thickness of 3 ft. 6 in. The minimum depth of concrete below the stone block invert was 12 in. The steel beams resting on top of the side walls were spaced 3 ft. apart on centers. The longest was 25 ft. and was of 24-in., 80-lb. section. The shortest was 18½ ft. long and was of 18-in., 55-lb. section. Figure 209 is a detail of the roof showing the construction. Both the form work and labor for a roof of this sort are liable to be less expensive than for masonry bellmouths, but the cost of the steel beams may affect the total cost of the structure so that it will not be as cheap as one of the older types.

There are certain theoretical features connected with the design of these junctions which should always be kept in mind, although it is a common experience that it is impossible to satisfy all theoretical requirements in work of this nature, and the best the engineer can do is to effect a compromise which will result in a structure of ample strength and fitness for the average demands of service. These theoretical considerations have been summed up by Frühling as follows:

Sewers must be joined in such a way that no decrease in velocity occurs, because that will result in the subsidence of the silt and suspended matter. It is as necessary to avoid, therefore, a widening of the channel as the formation of an obstruction to the flow, and the two channels should gradually blend into each other, but with the elongations and grades of the inverts so arranged that the discharges from the individual branches have the same surface elevation at the point of junction. With corresponding rising and falling of the sewage in the sewers which are brought together thus, it would be possible to base the designs on any proportion of the capacity of the sections being utilized, but as the surface of the sewage in the trunk sewer is ordinarily proportionally higher than that in the laterals, the engineer is compelled to select arbitrarily some proportion of the full capacity, as that which will be utilized, and to remember that an excess use of the capacity

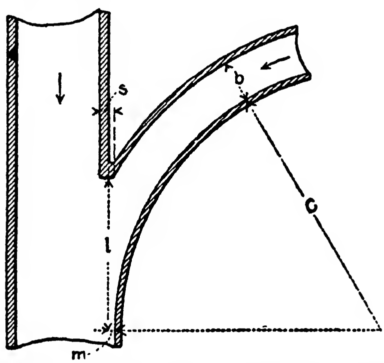


FIG. 210.—Symbols used in Frühling's discussion.

will cause the additional height in the trunk sewers to back up the sewage in the branches discharging into it (except those discharging close to its crown). The smaller the available difference in elevations, and hence the flatter the grades, the lower should be the proportion of the full capacity which is chosen as the basis of the design, but it must not be below a proportion which corresponds to the discharge of the average dry-weather sewage, in order that the backwater may be limited to the periods of flow of the maximum dry-weather sewage and of the storm water. With better grades, the design can be based on larger volumes of water, such as the maximum dry-weather discharge or a definite dilution of it by storm water; the upper limit corresponds to the runoff of heavy storms. In this case, assuming that the sewers run full, the crowns of the sewers are to be brought to the same elevation but the inverts will be at different elevations, depending upon the heights of the different sewers. In all lower stages, the sewage in the branches will enter the trunk sewer through a short section having an increased grade.

The length l (Fig. 210) of the junction, depenαs upon the radius r and the width b of the branch sewer, the increase in width m of the trunk sewer, and the thickness of the masonry s at the junction. Then

$$l^2 = (r + b + 0.5s)^2 - (r + m)^2$$

This shows that a change at the junction to a section of greater width, as from an egg shape to a semielliptical shape, reduces the length of the junction. So far as the value of r is concerned, it is never taken at less than $5b$ in the better class of designs; the resistance to the flow of the sewage increases as the radius decreases, but the resulting loss in fall is slight.

In large cities, the junctions are not always such simple affairs as those shown in Figs. 207 and 208. In Fig. 211 a complicated junction in Philadelphia is illustrated. Here there is a brick sewer 9 ft. in diameter crossing a brick sewer 8 ft. 3 in. in diameter, and the problem was to put in junction chambers and separate sewers in such a way that the course of the larger sewer, beyond this intersection, would serve as a relief for the storm water from the smaller sewer, and that the dry-weather sewage in the latter would flow into the channel which would also carry away the dry-weather sewage from the former. This was accomplished by four junction chambers and two 30-in. cast-iron pipe sewers, shown in the illustration. It will be observed that a very large proportion of the section of the 9-ft. sewer will be utilized before there is any discharge from it into the overflow sewer, while in the case of the 8¼-ft. sewer everything that is not dry-weather sewage will immediately be discharged into the overflow outlet.

Hydraulic Considerations.—Velocities of flow in sewers usually are not high enough so that it is necessary to give special consideration to the hydraulic features of transition structures at junctions. If reasonable care is taken to avoid sudden enlargements and contractions and to make changes of direction by smooth and reasonably flat curves, no difficulty is likely to develop. The condition is radically different, however, when high velocities are involved, say, of over 6 or 8 ft. per second. In such cases, the theoretical position of the flow line from point to point should be computed (by the application of Bernouilli's theorem) and the design of the structure should be changed, if necessary, to obtain a smooth curve for this line, tangent to the water surfaces at each end of the transition.[1]

SIPHONS

Unfortunately, there are not two words in the English language to make a sharp distinction between what we call inverted siphons, "Düker" in German, and true siphons, "Heber," in German. Consequently, engineers frequently speak of siphons when they mean inverted siphons, and considerable confusion sometimes arises on this account.

Where a conduit has a U-form in its profile between two points, that is to say, is provided with a descending and then a rising leg, it forms an

[1] Valuable suggestions will be found in a paper on "The Hydraulic Design of Flume and Siphon Transitions" by Julian Hinds, *Trans. Am. Soc. C. E.*, 1928; **92**, 1423.

inverted siphon. This may or may not have such a bend that the liquid in the bottom will always seal the legs like a trap. Where there is no such seal, the inverted siphon is commonly spoken of as "incomplete;" a complete inverted siphon is really a large trap, duplicating on a great scale the apparatus so familiar on a small scale in plumbing. The inverted siphon is therefore a length of sewer which is below the hydraulic grade line and consequently is under pressure.

A true siphon, on the other hand, consists of a rising leg followed by a falling leg, the two having an A-form and serving, by utilizing atmospheric pressure, to raise water above the hydraulic gradient between two points on a conduit. The siphon must discharge at a lower elevation than that at which the liquid enters it, and the maximum theoretical height over which the siphon is able to lift water is $(34 - h)$ ft., where h is the head in feet necessary to overcome friction and other resistances in the conduit. If air or gas collects at the summit of the siphon, it will eventually interrupt the service, and on this account various devices are used to guard against this danger. This is particularly important in the case of siphons operating with sewage, because of the tendency of gases to be given off by the sewage as the pressure to which it is subjected becomes reduced, toward the summit of the siphon.

Inverted Siphons.—In order to prevent the deposition of suspended matters in inverted siphons, it is desirable to maintain as high velocities as possible, say 2 to 3 ft. per second for domestic sewage and 4 to 5 ft. per second for storm water where conditions will permit. This is accomplished by confining the flow to one or more restricted channels and allowing a steeper hydraulic gradient through the siphon. In some cases catchbasins or grit chambers have been built just above siphons, but these are troublesome to clean and the material removed from them is usually offensive. The operation of siphons should be inspected regularly and, if necessary, they should be cleaned by flushing or scraping. Flushing is accomplished in various ways, depending upon the available facilities and surrounding conditions. If there is a pumping station above the siphon, flushing may sometimes be accomplished by increasing the pumping rate; automatic flush tanks may be installed at the head of the siphon; sewage may be backed up in manholes behind gates or stop planks and so discharged as to create a high velocity; or the opening of a blowoff at the low point of the siphon may induce the required velocity. The siphon may also be cleaned by rodding or scraping after drawing down the sewage through valves opening into a sump, as in Fig. 213.

Manholes or cleanout chambers should be provided at each end of a siphon, to give access for rodding, pumping, and, in the case of pipes of large size, for entrance. There is objection to the introduction of intermediate manholes on an inverted siphon in such a manner that the

sewage will be free to rise in them, since grease and other scum tends to fill up such a shaft with a solid plug. They are advantageous, however, if the sewage be confined within the siphon as it passes through the manhole, affording access or means of ridding the siphon of deposit, through a gated connection or similar device.

Since an inverted siphon is subjected at all points of its cross-section to an inner pressure, the walls may be in tension, although the tension may be neutralized by external water pressure or by the pressure of earth. On account of these tensile stresses, inverted siphons usually are constructed of steel, iron, reinforced-concrete, or wood-stave pipe heavily banded, though vitrified pipe has sometimes been used successfully under a small head or if the pipe be encased in concrete.

The computation of the sizes of pipe for inverted siphons is made in the same way as that of sewers and water mains. The diameter depends upon the grade and the maximum quantity of water to be carried. The latter is affected, in the case of inverted siphons under rivers, by the presence or absence of a storm overflow before the inverted siphon is reached, and on the degree of dilution of the sewage before the overflow outlet comes into service. The head or drop in the hydraulic gradient will equal the sum of the friction head, the head lost at bends and at changes in cross-section, and the loss in velocity head involved by the variations in velocity.

Practical experience has shown that where siphons are built of unduly large size they soon silt up to a point where the reduced section will increase the velocity of flow through it to a point sufficient to maintain the section. To overcome this difficulty, experience in this country has dictated the advantage of using several or multiple pipe lines, instead of one pipe for the siphon, arranged in such manner as to throw additional pipes progressively into action with increase in discharge of the sewage.

English authors have emphasized the importance of ventilating long inverted siphons, asserting that otherwise the flow may be interfered with by accumulations of air or gas, but American sewerage practice does not seem to have developed such difficulties.

Care must be taken that inverted siphons built on or under river beds have sufficient weight to prevent their flotation.

As an illustration of the principles of design, a problem is given that was encountered in the authors' practice, involving the design of an inverted siphon of several pipes to replace an existing single-pipe siphon which had given trouble from sedimentation due to low velocities. The length of siphon was 440 ft. with an available fall of 3.2 ft. and a maximum depression of 9 ft. below the hydraulic gradient. The 30-in. concrete sewer feeding the siphon is on a slope of 0.0037 and has a maximum capacity of about 25 cu. ft. per second. The minimum sewage

flow to be provided for is 4 cu. ft. per second, with a maximum dry-weather flow of 13 cu. ft. per second.

Allowing 0.4-ft. loss of head in the inlet chamber, the available slope for the siphon is 0.0064. Local conditions indicate economy in the use of vitrified pipe encased in concrete. Three pipes are provided: a 15-in. to carry the minimum flow, a 20-in. which in conjunction with the 15-in. will carry the maximum dry-weather flow, and a 22-in., the combined capacity of the three pipes being equal to the ultimate maximum that can reach the siphon.

A type of inlet chamber (Fig. 212) suitable in this instance has the invert of the pipe which carries the low flows continuous with that of the

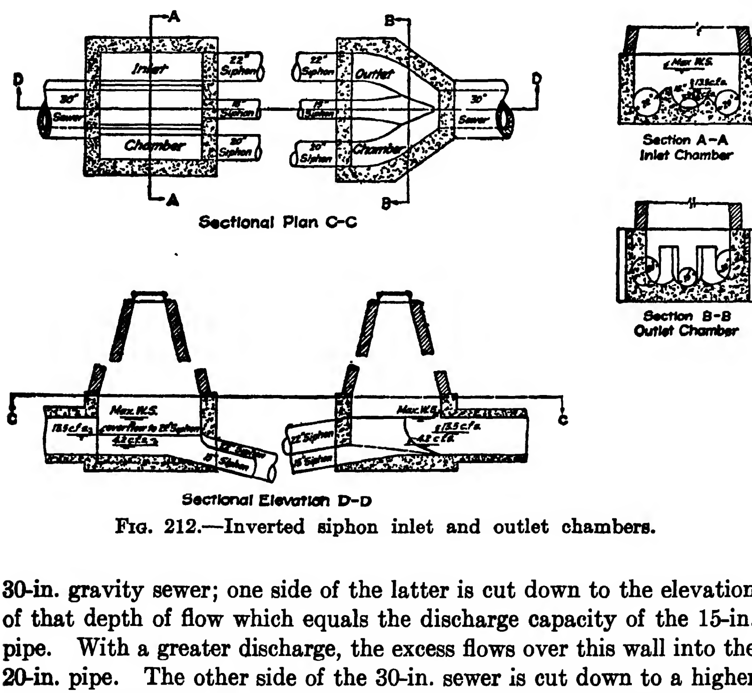


FIG. 212.—Inverted siphon inlet and outlet chambers.

30-in. gravity sewer; one side of the latter is cut down to the elevation of that depth of flow which equals the discharge capacity of the 15-in. pipe. With a greater discharge, the excess flows over this wall into the 20-in. pipe. The other side of the 30-in. sewer is cut down to a higher elevation representing the depth of flow in this sewer when the combined capacity of the 15- and 20-in. sewers is reached. Discharges in excess of this amount pass over this wall into the 22-in. pipe. The heights of these two walls are 8 and 16 in., respectively.

These sidewalls will be submerged during periods of maximum flow and should not be considered as weirs but as obstructions causing loss of head in passing the desired quantity of sewage over each. Since this flow is at right angles to the direction of flow in the approaching sewer,

this loss may be taken as the head required to produce the necessary velocity across the top of the wall, assuming the energy of velocity of approach to be lost in the change of direction. Assuming a length of overflow of 7 ft., the sewage to be carried by the 20-in. pipe will flow 8 in. deep over the wall and a velocity of 2.0 ft. per second is built up requiring 0.06-ft. head to produce it. After passing over the wall, building up the velocity of 4.3 ft. per second desired in the 20-in. pipe will require 0.29-ft. head, making a total head loss of 0.35 ft. while the assumed loss was 0.4 ft. Similiarly, the head loss over the wall, and building up 4.3 ft. per second velocity in the 22-in. sewer, will be

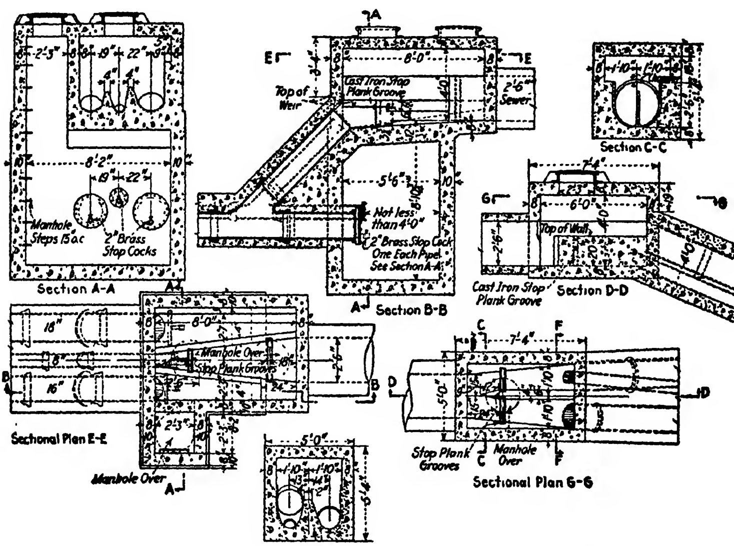


FIG. 213.—Inverted siphon on Middle Fork trunk sewer, Louisville, Ky.

$0.03 + 0.29 = 0.32$ ft., so that no revision of available slope through the siphon is necessary.

At the lower end of the siphon, the junction of the three siphon pipes with the 30-in. gravity sewer should be so designed as to reduce the opportunity for eddies to carry sediment back into those pipes which are not at the moment operating, but which are full of standing water. This is especially important in the case of the 22-in. pipe, which will not be in operation except at unusual rates of flow. It may be accomplished by maintaining the three pipes, or the corresponding channels within the junction chamber, as nearly up to the point of intersection as possible, to avoid pooling and reduction of velocity in the chamber. As a further precaution, the outlet of the 22-in. pipe (least frequently required) may

be raised so that the invert of its channel has a sharp forward pitch toward the intersection. The crown of the pipe must not, however, be raised above that of the 30-in. sewer, or it will lie above the hydraulic gradient.

This solution has been used in the outlet chamber of the siphon shown in Fig. 213. Here, the 18-in. pipe, which is the last to come into service, and which flows full only when the 30-in. sewer into which it discharges also is flowing full, is elevated till its crown is continuous with that of the 30-in. sewer. It would have been possible to elevate the 16-in. outlet similarly so that its crown elevation coincided with that of the water surface in the partially filled 30-in. section at the depth necessary to carry the combined discharges of the 8-in. and 16-in. pipes.

The inlet chamber includes a sump, or well, into which the siphons may be drained through 2-in. brass stop cocks, preparatory to removing the flanges should the pipes require unstopping or inspection. Stop planks in the inlet and outlet chambers prevent inflow of sewage during this operation. Where possible, at least 6 ft. clear headroom should be provided in both inlet and outlet chambers.

It sometimes becomes necessary to insert an inverted siphon in an existing sewer when some underground structure is to be built across its course. Frequently, a readjustment of the sewer gradient is not feasible and the additional head required will be derived from a rise in the upstream water surface. This increases the tendency toward deposition of solids in the sewer thus affected by backwater, and it may be necessary to require frequent inspections and flushing as occasion may demand. The construction of subways has necessitated the provision of many such inverted siphons which have been operated successfully. The first structure of this type, built in connection with the New York subways, was at 149th St. and Railroad Ave. and was placed in service in February, 1902. An example of this type of structure is seen in Fig. 214, which is taken from an admirable report[1] on "Inverted Siphons for Sewers." This siphon is on a combined sewer and provides a 12-in. cast-iron pipe for dry-weather flow and 48-in. circular concrete pipes for ordinary storm flows, while the excess at times of extreme flow is carried over the top of the subway by an overflow channel measuring 3 ft. 6 in. by 4 ft. 9 in.

Figure 215, which shows a structure on the sewerage system of Louisville, Ky., is introduced to illustrate the manner in which a bypass can be constructed to discharge the sewage into a neighboring creek or other body of water when the inverted siphon requires cleaning. This structure includes two 12-in. iron pipes carried under the creek on the same invert grade as that of the sewer. In addition to these, there is provided a 36-in. iron pipe dipping down from the grade of the sewer

[1] *Jour. Boston Soc. C. E.*, 1921; 8, 237; also *Munic. Engrs. Jour.*, March, 1917.

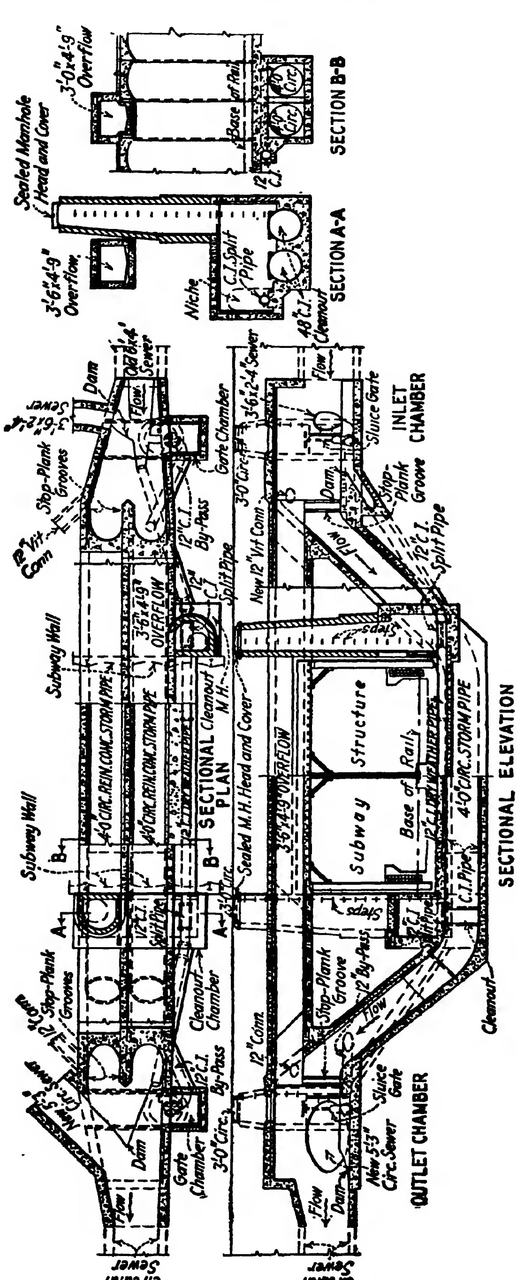


FIG. 214.—Inverted siphon under subway at Martense St. and Nostrand Ave., Brooklyn, N. Y.

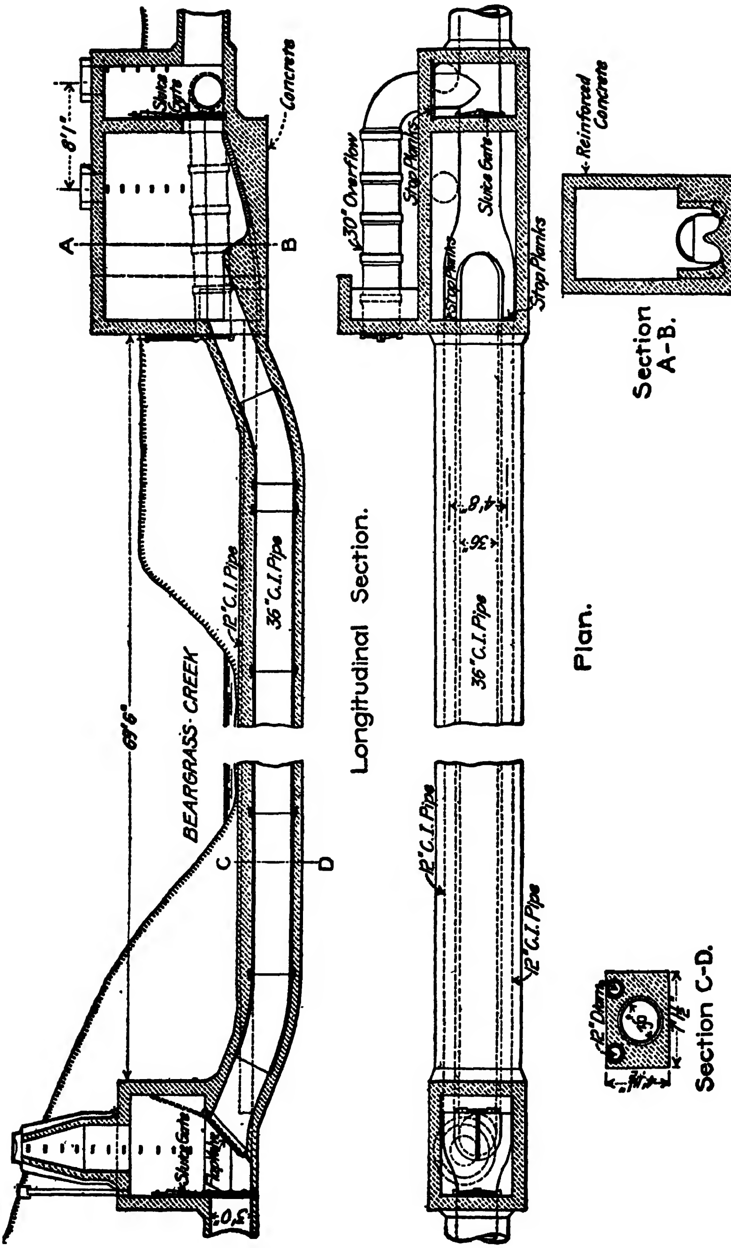


Fig. 215.—Inverted siphon at Louisville, Ky.

beneath the bottom of the creek. This pipe will act as an inverted siphon, but will not be put into use until the flow in the sewer exceeds the combined capacity of the two 12-in. pipes. At each end of the crossing there is a concrete chamber giving access to the siphon to facilitate cleaning when it is found to be necessary. There is also an emergency outlet to the creek, through which the sewage may be turned when the siphon is being cleaned or repaired. A sluice gate in the outlet chamber makes it possible to shut off any backwater from the intercepter at such times. The concrete protection of the pipes has its top on the level of the bottom of the creek.

A longer structure, also on the Louisville sewerage system, is shown in Fig. 216. This is on the line of a 48-in. sewer and consists of vitrified pipe encased in concrete. At the inlet chamber, the arrangements are such that any one or two or all of the pipes may be put in service, according to the quantity of sewage flowing. It was the intention of the designer to confine the entire flow to the 18-in. pipe so long as the quantity of sewage did not exceed its capacity, and then to substitute one of the 30-in. pipes. Other changes can be made from time to time so as to provide the necessary increase in capacity to meet the growth of the city. The entrance to each pipe is controlled by a sluice gate set in the masonry and also by stop planks and overflow chambers, so that in case of emergency the sewage will flow automatically into a second or third pipe when the one in use is overcharged. Provision is also made for an automatic overflow into the neighboring creek, and if it is necessary the entire discharge of the sewer may be turned for a short time through a 30-in. blowoff conduit into the creek. At the outlet chamber, any or all of the pipes can be closed by means of stop planks.

At the lowest point of this siphon a third chamber is provided for the purpose of draining and cleaning any of the pipes. For this purpose, the sewage is drawn off into a sump and then pumped into the creek, after which a section of the pipe 4 ft. long can be removed and the line running from it to either chamber can be cleaned in the usual way.

There are several river crossings on the sewerage system of Concord, Mass., each consisting of a line of 12-in. cast-iron pipe. At the head of two of these there are flushing chambers for accumulating sewage and discharging it intermittently in large quantities in order to keep the pipe clean. Each chamber is built of brickwork and has a dome roof as seen in Fig. 217; one is 20½ ft. in diameter and discharges from fifteen to twenty times in 24 hours, and the other is 10½ ft. in diameter and discharges eight to ten times in 24 hours. The chambers are emptied by means of Van Vranken automatic siphons.

There are a number of inverted siphons crossing Paxton Creek in Harrisburg, Pa., in order to deliver sewage to an intercepter built in 1903 from the plans of James H. Fuertes. The connections at both ends

of these siphons are shown in Fig. 218.[1] At the inlet end of each, a section of the existing sewer was taken out of sufficient length to permit the construction of a new manhole, sump, and connection with a silt basin. The dry-weather sewage as it comes down the old sewer runs down a cast-iron pipe leading from the sump in the sewer invert to the silt basin, which has a depth depending upon the conditions encountered at each crossing. The two outlets from this basin are 3½ and 4½ ft. above its bottom, and the sewage flows through them and down under the creek in two lines of cast-iron pipe, rising at the other side in a shallow manhole, from which it is discharged into the intercepter through a cast-iron drop pipe. The sewers are on the combined system, and the entrances to the inverted siphons were designed to permit the greater part of the storm flow to pass directly into the creek through the old outlets, flap gates being provided just beyond the sump to prevent backwater from entering the intercepter in times of flood. These gates were made of cypress lumber in order to secure lightness, and were faced with rings of steel where they bore upon the cast-iron frame. Each gate was hung on two wrought-iron straps extending its entire width and sunk into the lower side of it. After the gate had been hung and closed, the face joint was made by pouring lead into a groove left in the face of the frame for that purpose.

The sump at the intake end of the inverted siphon, through which the sewage enters the silt basin, is protected by a cast-iron grating. The outlets from the silt basins are provided with cast-iron hoods to prevent floating matter from getting into the inverted siphon, and sluice gates at the bottom of the basin afford means for cleaning the pipes. The two pipes under the creek unite at the discharge end in a manhole, from which the sewage flows down a cast-iron pipe into the intercepter.

An inverted siphon shaped like a Venturi meter to prevent deposition of suspended matter by an increase of velocity without appreciable loss of head, has been built on the Thirty-ninth Street conduit under the Illinois and Michigan Canal in Chicago. The top of the conduit required lowering 10 ft. to permit the necessary 4 ft. 8 in. of water in the canal. The section of the conduit was gradually changed from an ellipse on end, 14 ft. high and 12 ft. wide, to one on its side, 9 ft. high, and then back again. The throat section of the siphon is about 65 per cent of the full section, and it was estimated that the loss of head would be less than 0.03 ft., while the velocity would be increased to about 3 ft. per second, which was considered a transporting velocity for the material likely to reach that point. The inverted siphon is 200 ft. long, and constructed of 12-in. thick, 1:2½:5 concrete reinforced with ½-in. steel rods forming hoops 6 in. apart and longitudinal ties 12 in. apart.

[1] *Eng. Record*, 1902; **46**, 341.

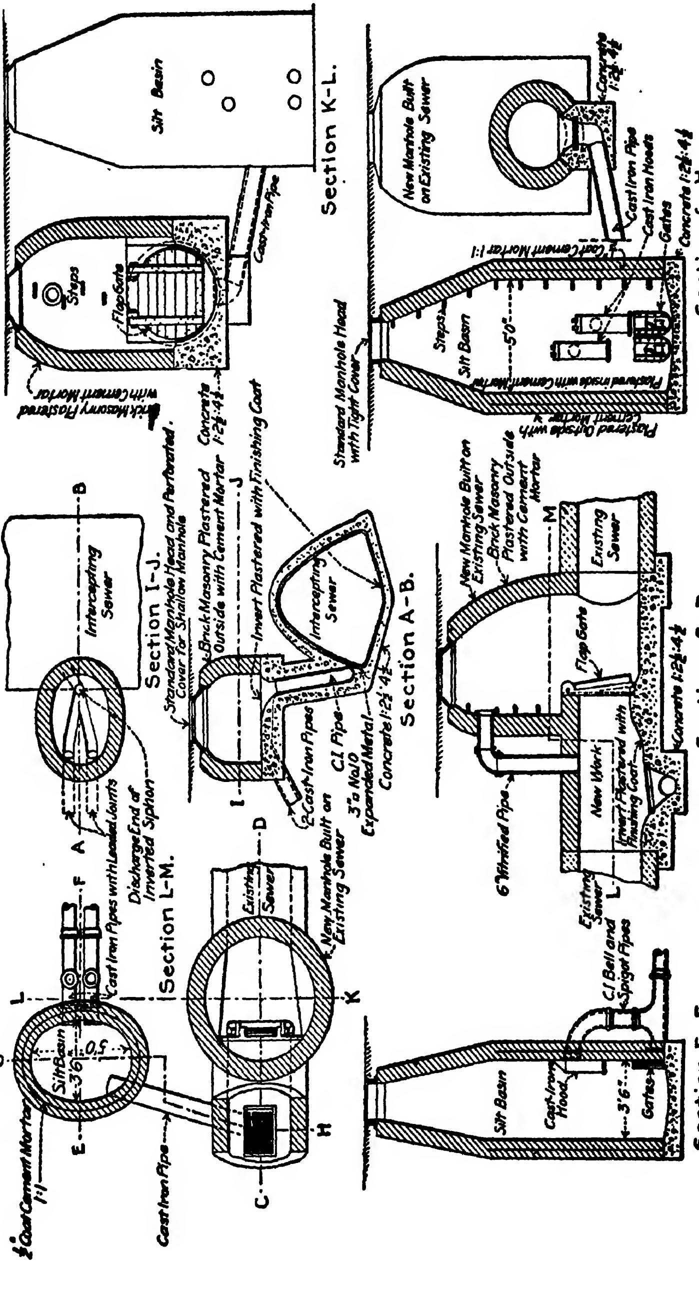


FIG. 218.—Inverted siphon at Harrisburg, Pa.

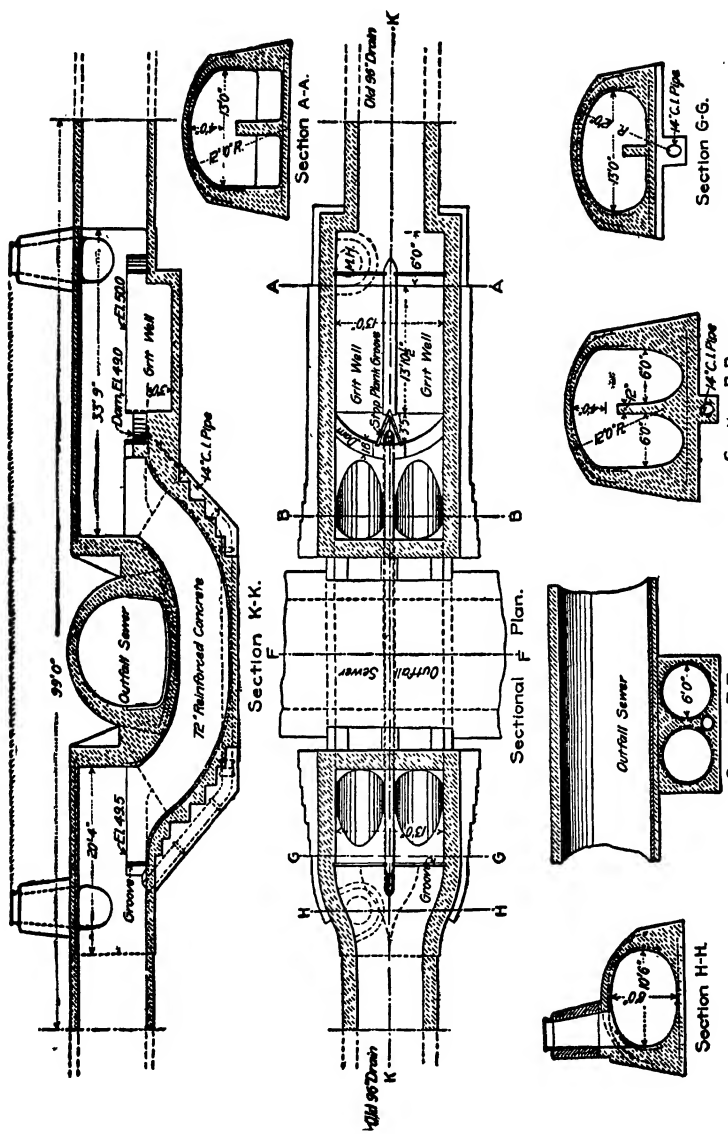


FIG. 219.—Inverted siphon on storm-water drain, Baltimore, Md.

A typical short inverted siphon is shown in Fig. 219. It carries the contents of a large drain under the outfall sewer at Baltimore, and has two 6-ft. circular conduits and one 14-in. pipe, all ending on each side in an enlarged chamber. The upstream end contains two grit wells, separated by a wall rising 2½ ft. above the bottom of the invert of the sewer. Either grit well can be shut from the sewer at the height of this division wall by means of stop planks. The ends of the grit wells toward the inverted siphons are closed by curved dams, the tops of which are 1 ft. below the top of the division wall. The 14-in. cast-iron pipe is in the center line of the sewer and can be shut off from connection with either grit well by stop planks between the division wall and the end of the curved dam. The intake of the 14-in. pipe is 1½ ft. below the crest of the dam and 2½ ft. below the top of the division wall. In ordinary operation the pipe carries the flows, and when these exceed its capacity, the sewer discharges into one of the 6-ft. siphon pipes over the dam at the end of one of the grit wells. Should the storm become more severe, the 14-in. pipe and both 6-ft. conduits will be put in operation. The normal arrangement is to place stop planks across one end of one of the grit wells and between this grit well and the opening of the 14-in. pipe. The sewage then flows unobstructedly to the other grit well and into the 14-in. pipe. When it rises high enough, it overtops the dam at the end of the grit well left open, and, when it rises 1 ft. higher, overflows the stop planks at the head of the other grit well and division wall and discharges through both of the inverted siphons. On account of the custom of sweeping street refuse into the storm-water drains, the structure is required to work under trying conditions, but it has operated successfully since its completion.

True Siphons.—One of the oldest true siphons in a sewerage system crosses the St. Martin canal in Paris. At this place there is a masonry arch bridge, the Pont Morland, and the siphon is attached to one face of it, forming a semicircle with a diameter of 52½ ft. Its crown is a little more than 26¼ ft. above the sewer leading to it. The gases which are given off from the sewage rise to the top and are led away through a riser 49.2 ft. high, from which they are drawn by an ejector worked by water admitted and shut off at the right times by a float. The siphon can be put in operation by means of the ejector in about 5 min., so that any serious interruption in its service is regarded as unlikely. French engineers have made tests of this siphon, which have shown the surprising fact that with a velocity of flow of 3.9 to 4.9 ft. per second, the collection of gases at the crown no longer takes place.

It is generally believed that the first sewage siphon in the United States was constructed at Norfolk, Va., about 1885. It is a cast-iron line 14 in. in diameter, and about 1,900 ft. long, which was built by City Engineer W. T. Brooke to avoid troublesome and expensive trench work

TABLE 161.—DATA RELATING TO A FEW OF THE LARGER OR LONGER INVERTED SIPHONS[1]

Location	System	Obstacle	Number and size of conduits	Length, feet	Material	Authority
Buenos Aires, Argentina	Separate and combined	River Riachuelo	1–9.65′	443	Tunnel with cast-iron segment lining.	*Eng. Contr.* 1919; **51**, 161.
Paris, France.—Clichy	....	River Seine	1–7.54′	1,519	Grouted tunnel, cast-iron lining	*Eng. Record*, **32**, 409.
Los Angeles, Calif., Sec. 3	Combined	Valley	1–38″	16,936	Wood-stave pipe	*Eng. News*, **33**, 142.
Los Angeles, Calif., Sec. 6	Combined	Valley	1–36″	17,174		
Milwaukee, Wis., High Level	Combined	Harbor	1–48″	6,482	Cast iron and concrete	Sewerage Comm. drawings, T. Chalkley Hatton, Chief Engineer.
High level	Combined	Harbor	1–51″, 60″, 67″, 72″	6,439	Cast iron and concrete	
High level	Combined	River	1–6′-0 to 6′-9 ±	4,874	Cast iron and concrete	
Boston, Mass.	Combined	Dorchester Bay	1–90″	7,160	Brick-lined tunnel	"Main Drainage Works of the City of Boston," Eliot C. Clarke, 1885.
Fitchburg, Mass.	Combined	Nashua River	1–30″	5,300 ±	Cast iron	H. P. Eddy, Consulting Engineer
Charlestown, Mass.	Combined	Mystic River	1–57.5″	1,085	Brick	Metropolitan Water and Sewerage Board.
Worcester, Mass.	Combined	Low ground	1–72″	1,500 ±	Reinforced concrete	Ralph G. Lingley, City Engineer.

[1] Compiled except last item from report on "Inverted Siphons for Sewers," *Jour. Boston Soc. C. E.*, 1921; viii, 237.

in quicksand. The outlet end is provided with a return bend, which prevents the siphon from becoming unsealed, and at the summit there is attached a 2½-in. pipe through which accumulations of gases and air are removed by means of an air pump at the sewage pumping station. In regard to the operation of this siphon Norman Z. Ball, Engineer, Division of Water and Sewers of Norfolk writes (1927) as follows:

The only trouble we have had in the operation of this sewer of which I know, occurred about 3 years ago. We had trouble maintaining our vacuum on the siphon so we installed a new vacuum pipe but this failed to correct the trouble. We then began sounding along the main siphon and eventually located two points at which air was leaking into the siphon. On taking out these two sections we found that the sand flowing along the bottom of the pipe with the sewage had worn a V-shaped groove entirely through the cast iron. On replacing these damaged sections of pipe, the siphon was restored to service and has worked satisfactorily ever since.

The best-known siphon is probably that constructed at Breslau in 1885, to carry the sewage of a population of about 5,000 people from an island in the Oder to the right bank of that river. It is hung from the superstructure of a bridge and is 493.6 ft. long and 5.9 in. in diameter. The highest point of the siphon is at the end of the bridge, and from it the descending leg drops down into a water seal in the bottom of a manhole. At the summit there is a chamber in which the gases are collected. As these gather, the level of the sewage in the chamber gradually falls and finally it reaches such a point that a float inside the chamber operates a water-driven ejector, which sucks off the gases and is finally closed by the rising of the float. This siphon, which was the first of several of the same type in Breslau, although expensive, proved an economical substitute for an inverted siphon which would have been very costly on account of local conditions.

Extensive use is made of siphons in Potsdam, where one of them has been employed, in fact, as an intercepting sewer. At each point of interception the dry-weather sewage is discharged into a chamber, where it first deposits any silt or sediment in a sump, and then passes over a wall and through a screen into the bottom of the rising leg of a siphon. At the mouth of this siphon there is a sliding valve operated by a float, and somewhat higher in the rising leg there is a ball valve. The float-valve closes the siphon whenever there is a chance that the sump will be drained completely of sewage, and the ball valve is an assurance against the entrance of air. The gases and air are forced out of the siphon by water injected under pressure into the summit. In order to accomplish this the two legs of the siphon must be closed, which is done by means of the valves already mentioned at the inlet end, while at the outlet end, which is at a pumping station, a valve is shut by the attendant before he

admits the water under pressure into the siphon pipe. The details of the air-removing chamber at the summit have been worked out so that as the gases are put under a fairly heavy pressure, they lift a heavy valve and escape through small openings into the air. As they escape a float rises on the liquid which replaces the air. This float carries a vertical rod with a needle point at its upper end. When the float has risen to the maximum position, this needle point enters the orifice through which the gases escape, and closes it. This closes the passage so that the heavy valve at the top of the passage falls back on its seat. The attendant at the pumping station observes, by means of a pressure gage, when this takes place, and shuts down the machinery which puts the siphon under pressure. There are three points where intercepting sewers discharge into one of these siphons on the Potsdam sewerage system. A description is given in Frühling's "Entwässerung der Städte."

SEWER BRIDGES

The use of bridges in connection with sewers has been fairly infrequent, particularly in the United States. The difficulty has been the strong objection to the use of true siphons for such crossings, and the fact that it is rarely possible to support a sewer from a bridge structure unless it is carried up from its position in the street to about the level of the roadway of the bridge, thus forming a siphon.

A river crossing on the joint outlet sewer in northeastern New Jersey, built from the plans of Alexander Potter, is shown in Fig. 220. This is such an elementary structure that it is hardly suitable to speak of it as a bridge. The 42-in. cast-iron pipe is supported on posts made of pairs of rails embedded in concrete piers 4 ft. deep and 7 by 4 ft. in plan. There is one of these supports for each length of pipe. This construction was employed in order to minimize the obstruction to the stream flow and secure the greatest possible clearance between supports. It will be observed that the river channel at this place was widened considerably so as to afford a greater waterway. A more elaborate structure on the same sewerage system is shown in Fig. 221.

A reinforced-concrete sewer bridge was constructed at Morristown, N. J., to carry a 2-ft. sewer across a stream at a point at which there was not sufficient head available to permit the use of an inverted siphon. The stream is flashy and consequently the channel was widened at the site of the bridge and the crossing was made in three spans of 33 ft. each, giving a clear width of 99 ft. There was some possibility that the structure might be widened and used as a highway bridge, and accordingly the girders were made heavier than would otherwise have been the case. The cross-section of the bridge has a width of 4 ft. and a depth of

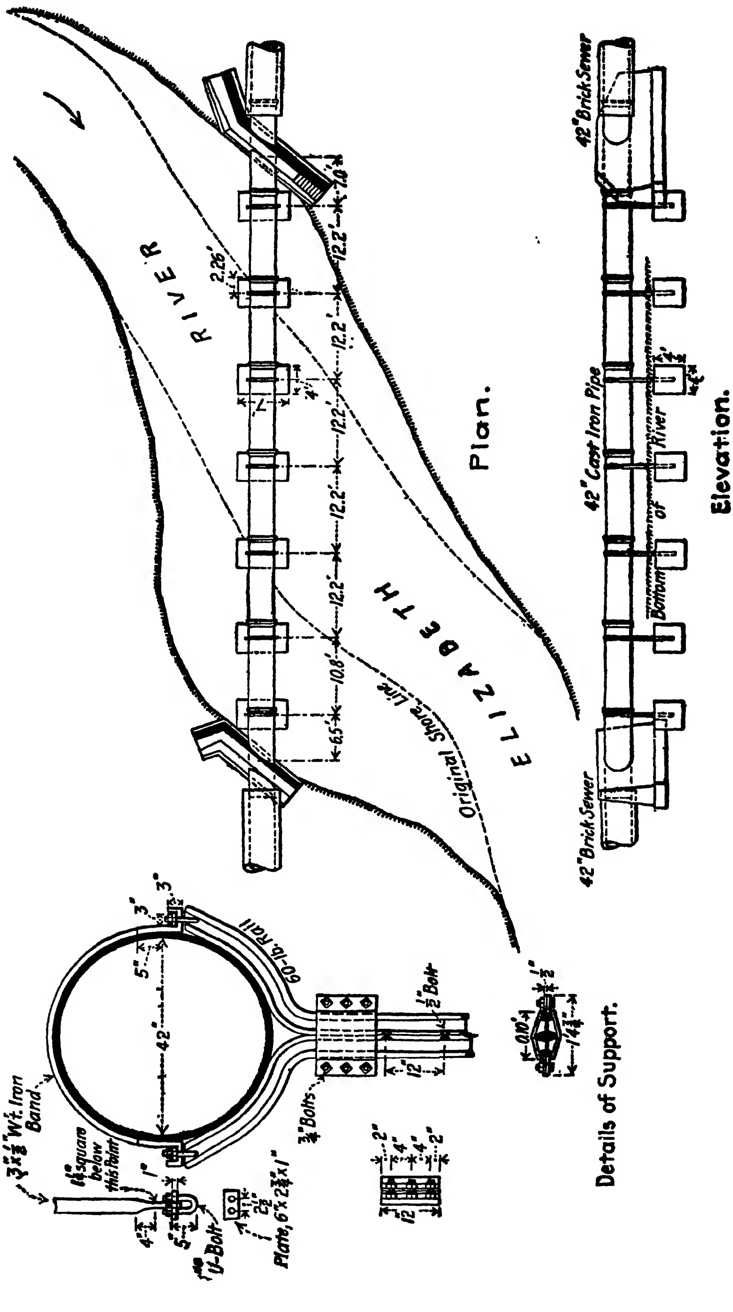


FIG. 220.—Sewer bridge, joint outlet sewer, New Jersey.

32 in. The 2-ft. sewer is in the center. This permits the design to be regarded as a pair of girders 12 in. wide and 32 in. deep. This bridge is said to have cost about 20 per cent less than the bids for a structure consisting of an iron pipe suspended between steel girders.

A 4½-ft. sewer is carried across a canal in Denver, Colo., by means of a reinforced-concrete bridge, 44 ft. long, with a clear span of 40 ft. In cross-section it is 4 ft. 8 in. high and 7 ft. 6 in. wide. The circular 4½-ft. sewer is located so that there is 6 in. of concrete below the vitrified-brick invert. This gives a cover of about 5 in. above the crown of the section, the top of the bridge having a transverse slope, each way from the center, of about 1 in. The structure is reinforced on each side of the sewer as if both sides were beams, and the total

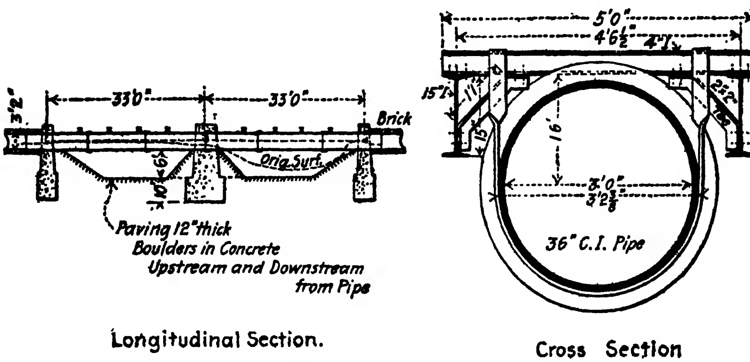


FIG. 221.—Sewer bridge, joint outlet sewer, New Jersey.

dead load of the span is 93 tons. The design was made by H. F. Meryweather,[1] who considers that a needlessly heavy and strong structure was built.

A reinforced-concrete structure of a somewhat lighter character was built in Los Angeles in 1907, to carry a 36-in. cast-iron pipe sewer across the Los Angeles River. On each side of the pipe is an 18-in. 55-lb. steel I-beam wrapped thoroughly with 3/16-in. wire surrounded with concrete. Every 36 ft. these beams rest on a reinforced-concrete pier which is supported on two reinforced-concrete piles. Every 12 ft. the pair of beams are connected by a reinforced-concrete diaphragm which forms a support for the pipe.

A box-girder sewer approximately 22 in. wide and 34 in. high was constructed in 1910 in St. Louis, across a ravine which it was expected to fill within a few years, but it would take so much time for the fill to settle thoroughly that it was deemed inadvisable to delay the construc-

[1] *Eng. News*, 1912; 68, 272.

tion of the sewer on that account. The design adopted for this project was a hollow concrete girder of two 35-ft. spans with a central pier.[1] The girder was designed to carry the weight of the concrete, the sewage, and a triangle of earth on top of the sewer, 3 ft. high. This last provision was to allow for the load which might come on the sewer when the ravine was being filled and before the fill had compacted enough to carry the weight of the sewer.

FLUSHING DEVICES

The primary purpose of flushing is to permit sewers to be laid on flat grades which, while producing adequate velocity to give the desired capacities at the depths assumed in the computations, are not enough to give at other depths velocities which will carry off at all times all solid matter. The problem of flushing, strictly speaking, is usually merely one of keeping lateral sewers clean from their dead ends to the points where the flow of sewage is great enough to accomplish this without assistance from the water mains. Occasionally the problem is one of furnishing a large volume of water to clean out a main sewer or an inverted siphon. In any case, the object is to increase temporarily the hydraulic gradient in the sewer by means of an exceptional head of water at its upper end. In some European cities, the volume of water stored for flushing is quite large, so as to maintain the discharge under this extra head for a considerable period; in the United States, the quantity stored in a flush tank at the end of a lateral sewer is not usually over 350 gal.

Flushing from Brooks.—An example of flushing a large sewer from a neighboring water course is afforded by the intake on the Harbor Brook Intercepter in Syracuse, designed by Glenn D. Holmes and shown in Fig. 222. The sides and bottom of a brook near this sewer were paved with concrete, and provision was made for temporarily damming the channel with stop planks. The water thus impounded can be diverted through two 15-in. vitrified intake pipes surrounded by concrete, into an 18-in. manhole. This is built of concrete with a vitrified pipe as the shaft, its bell being closed, when not in use, with a stop plank of two thicknesses of 1-in. pine, which can be lifted out of the bell by a chain when flushing is to begin. From the bottom of this manhole, a 24-in. vitrified pipe runs directly into the 33-in. circular concrete intercepting sewer. A manhole is located a few feet farther up the line of the sewer. The difference in elevation between the top of the temporary stop planks in the creek and the invert of the intercepter is about 8 ft.

A flushing manhole built on the Minneapolis sewerage system (1895) from the designs of Carl Ilstrup,[2] is shown in Fig. 223. This manhole

[1] *Eng. News*, 1912; **68**, 426.
[2] *Eng. Record*, 1896; **33**, 296.

was constructed where a large brick sewer crosses a swamp and in so doing runs through a creek. The ground was very soft and troublesome, and piles were driven on which a grillage was laid below the lowest

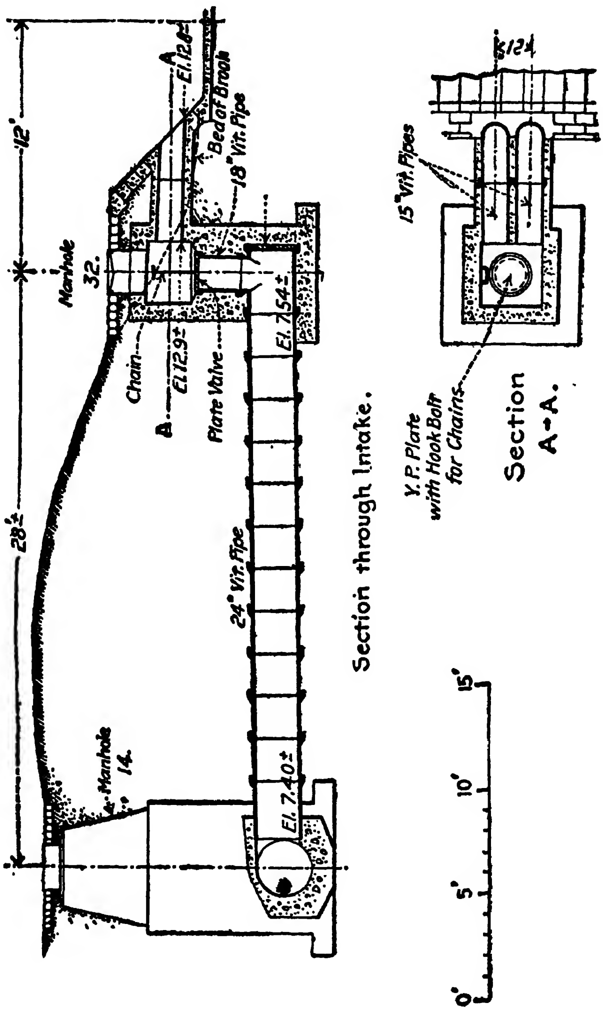


FIG. 222.—Flushing intake, Syracuse, N. Y.

water level, affording an opportunity to build the sewer inside a coffer. Stone walls were first laid and afterward a mass of concrete of sufficient volume to give the necessary weight for such a structure, was placed between them. At one side of the sewer, the excavation was extended

sufficiently to deepen the bed of the creek into a shallow well, which was roughly walled and paved so as to bring its bottom about on a level with the springing line of the brick arch of the sewer. The manhole built up on this foundation had a 2-ft. opening into the sewer, which could be closed tightly by a sliding door. On the opposite side of the manhole was an opening into the creek guarded by iron bars to keep out rubbish. In this way the manhole was kept full of water up to the level of the surface of the creek, and, whenever it was desired to flush the sewer, the sliding gate between the manhole and the sewer could be opened, admitting creek water under a small head.

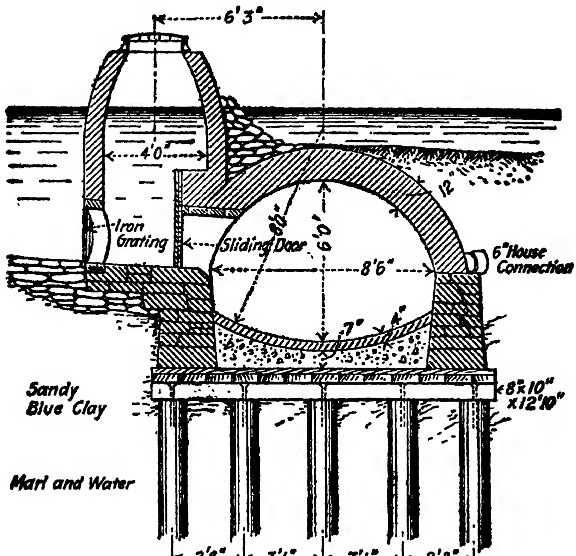


FIG. 223.—Flushing manhole, Minneapolis, Minn.

A flushing chamber was built at the end of an intercepter constructed in Harrisburg, Pa., in 1903, from the plans of James H. Fuertes, and worked satisfactorily for a considerable time, but was finally practically dispensed with, owing to the admission of brook water at a manhole a short distance below the headworks. It was necessary to use very flat grades in order to avoid prohibitive excavation and pumping, and this grade difficulty was overcome by making the sewer somewhat larger than necessary for the interception of the dry-weather sewage alone, and by forming a connection between the upper end of the sewer and the neighboring creek, where automatic regulating gates admitted during dry weather enough creek water into the conduit to keep the flow in all its parts at a self-flushing velocity. During storms, when the lateral

sewers were discharging large quantities of both sewage and street water into the intercepter, a float rose which closed the valve and shut out the creek water.

The design of the chamber is shown in Fig. 224. Two sets of three 12-in. vitrified pipes extend through the concrete headwall as inlets for the creek water, one set 4 ft. higher than the other. The water passes

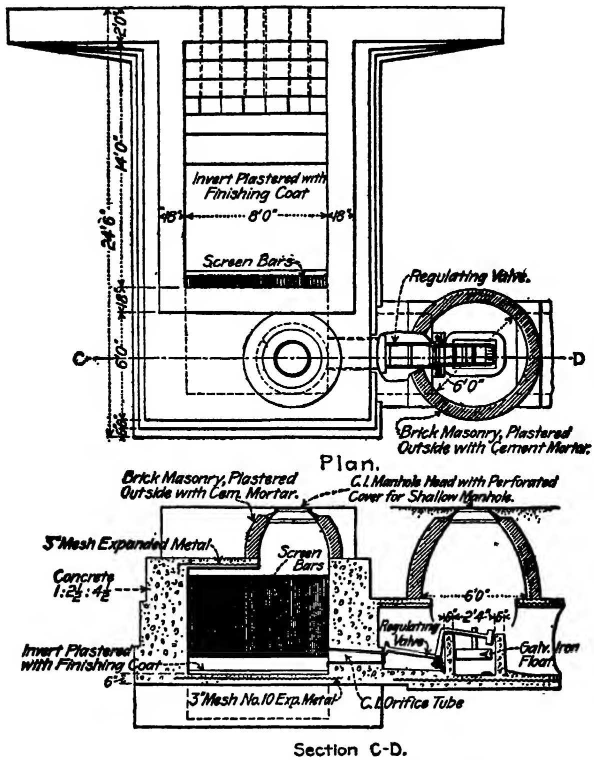


FIG. 224.—Flushing chamber, Harrisburg, Pa.

through a large silt basin in order to become free from heavy suspended matter, and then passes through rectangular cast-iron orifices into the regulating chamber proper. The valve regulating the admission of the creek water is of the usual type, the opening being automatically controlled by a galvanized iron float. The rotating arm is attached to the concrete wall of the float well by a short length of angle iron, the hole through which it is bolted to the latter being slotted so as to permit a vertical adjustment. The horizontal leg of this angle and the flanges of the trunnion carrying the rotating arm are slotted to allow a hori-

zontal play in two directions. When the valve was installed it was loosely bolted in place, adjusted by means of the slotted holes until it worked perfectly, and then bolted to its final position.

The float well was connected to the sewer by a 4-in. vitrified pipe extending below the invert about 10 ft. down the sewer, where the opening was covered by a cast-iron grating cemented into the bell of the pipe. All parts of the valve with its rotating arm and lever were of cast iron except the face of the valve and all wearing parts, which were bronze. The galvanized iron float was 11 by 24 in. and 9 in. deep. With the exception of the brick manhole the entire construction was of 1:2½:4½ concrete reinforced by 3-in. No. 10 expanded metal.

In Europe sewers are occasionally flushed by means of the sewage itself. To accomplish this, flushing chambers which contain large gates are employed. These gates are usually open, but are closed when flushing is to be undertaken. After they are closed the sewage backs up behind them and when a sufficient quantity has been stored it is suddenly released by opening the gates, which is accomplished in a variety of ways. Apparatus of this nature has rarely been proposed in the United States. Other methods of keeping the sewers clean are generally preferred and are described in Vol. II.

Flushing Manholes.—The flushing of small sewers is carried on either by hand or with the help of automatic apparatus. Opinion seems to be divided regarding the merits of the two methods; the authors' views are stated in Vol. II, under the operation of sewerage systems. As a general proposition, all flush tanks require some maintenance, and their cost is therefore dependent, in a measure, upon the time spent in inspecting and repairing them. The cost of this time, plus the interest and depreciation on the investment in the apparatus, plus the cost of the water used for the flushing, must be offset against the cost of labor and water where hand flushing is employed, for the difference in the cost of the manholes used in the two cases is negligible. The amount of water to be used for flushing and the frequency of the flushing depend not only upon the grade of the sewer to be kept clean, but also upon the possibility of dirt finding its way into the sewer.

Hand flushing is generally done by means of a hose from the nearest fire hydrant, inserted into the manhole at the end of the lateral or on the summit of the sewer to be cleaned. Flushing manholes are also used to a considerable extent. In this case a 1- or 1½-in. branch from the nearest water main is run into the manhole and the entrance to the sewer can be closed with a flap or tripping valve. Water is admitted to the manhole through the service pipe, and when it is full the valve is tripped, allowing the water to rush into the sewer. The same end is accomplished in some places where valves are not used, by plugging the end of the sewer with a disk consisting of sheet rubber faced with

canvas and held firmly between boards about ½ in. smaller than the diameter of the sewer. When the tank is filled with water this plug is drawn out, thus starting the flush.

Automatic Flush Tanks.—The flushing done with automatic apparatus is generally much more frequent than where hand flushing is

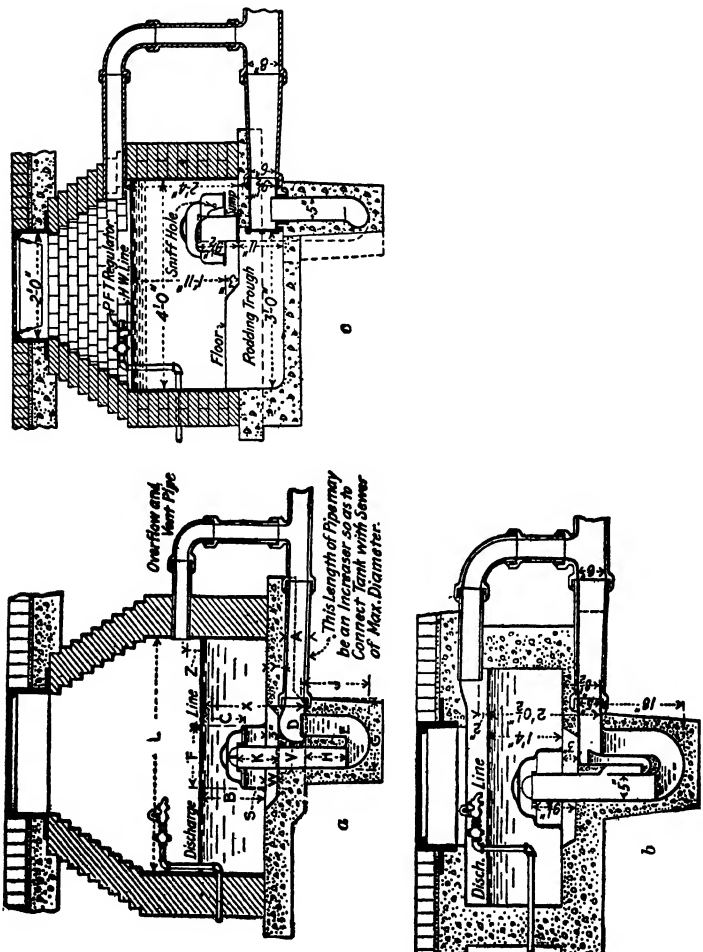


FIG. 225.—Different types of flush-tanks. (*Miller.*)

practiced, a common rule being to discharge the flush tank once every 24 hr. The water is usually admitted to these tanks through special orifices, of which a variety are manufactured by the makers of flushing siphons, so that any desired rate of flow under any street main pressure

can be attained by screwing the proper orifice or nozzle into the end of the service pipe. As a rule these jets are also accompanied by a mud drum or screening device and a blowoff cock, provided to insure the nozzle against clogging.

The operation of a siphon of the simplest type is as follows: In Fig. 225*a*, the Miller siphon is shown just about to discharge. There are two volumes of water separated by the compressed air in the long leg *V* of the trap. As the pressure on every part of this confined mass of air must be equal to the hydrostatic pressure, and as there are but two places where water is in contact with the air, it follows that the depth of water *C* in the tank must be the same as the depth *ED* in the trap. When the depth *C* is increased the water flows over the raised lip of the trap at *D*, this discharge allowing a little air to escape below the bend at *E*. The air pressure being released in this way, water passes up with a rush within the bell and into the long leg of the trap, and starts the siphon, the remaining air being carried off with the water.

The elevation of the lip of the short leg at *D* above the bottom of the outlet is an important detail, as upon it the first sudden discharge of the trap seems to depend. In the older types of flushing apparatus this first strong flush was accomplished by using an auxiliary siphon at the bottom of the trap casting, a detail retained in the Rhoads-Miller siphon, Fig. 225*b*, for use where shallow construction is imperative.

When the water in the tank has been drawn down until its surface is below the sniff hole *S*, some air enters the bell and slows up the siphonic action but the discharge continues until the water reaches the bottom of the bell, when the water in the two legs of the trap forms a seal and the tank begins to fill again.

The dimensions of the Miller apparatus, required by designers, are given in Tables 162, 163, and 164. The diameter of the tank is the minimum which is generally considered desirable for siphons of the sizes listed. The discharge is the average given by the makers for that size and setting of siphon.

The setting shown in Fig. 225*a* does not afford access to the sewer, but the Miller-Potter design shown in Fig. 225*c* overcomes this defect. The manhole at the dead end of the sewer is provided with a flush tank and siphon, and while this is more expensive than the standard type, it affords an opportunity to insert a cleaning rod into the end of the sewer.

These and several other patterns of Miller siphons are made by the Pacific Flush Tank Company, who also furnish a "flush-tank regulator" having a screen and a glass orifice, so as to guard against clogging of the small orifice by foreign matter or by corrosion, or its enlargement by wear, either of which will interfere with the regular and proper operation of the flush tank. This regulator is shown in Fig. 226.

TABLE 162.—DIMENSIONS AND CAPACITIES OF MILLER SIPHONS, STANDARD SETTING (FIG. 225*a*)

Sewer diameter, inches A	Average discharge rate, cubic feet per second	Bell diameter, inches F	Trap width, inches G	Trap depth, inches J	Trap rise, inches K	Minimum tank diameter, feet L	Depth, inches, from discharge line to Floor B	Depth, inches, from discharge line to Invert X	Floor depth, inches Y	Rise to overflow, inches Z	Trap diameter, inches V
4 to 6	0.35	13¼	14	14¼	8¾	3	14	22¼	4¼	3	4
6 to 8	0.73	16⅜	18¼	22¾	9½	3	23	34	5	2	5
8 to 10	1.06	20½	20⅜	29⅝	11	4	30	44	6	2	6
12 to 15	2.12	25½	27½	36½	13	4	35	51¼	6¼	2	8

TABLE 163.—DIMENSIONS AND CAPACITIES OF MILLER SIPHONS, SHALLOW SETTING (FIG. 225*b*)

A, inches	Average discharge, cubic feet per second	F, inches	G, inches	J, inches	K, inches	L, feet	B, inches	X, inches	Y, inches	Z, inches	V, inches
6 to 8	0.55	16⅜	21	18	9½	3	16	24½	3½	2	5
8 to 10	0.90	20½	20	19¼	11	4	19	39	6	2	6

TABLE 164.—DIMENSIONS AND CAPACITIES OF MILLER-POTTER SIPHONS (FIG. 225*c*)

A, inches	Discharge, cubic feet per second	F, inches	G, inches	J, inches	K, inches	Minimum L, feet	B, inches	X, inches	Y, inches	Z, inches	V, inches
6 to 8	0.73	16⅜	18¼	22¾	9½	3	23	34	5	2	5
8 to 10	1.06	22½	20⅜	29¾	11	4	30	44	6	2	6

A counter by which the number of times the flush tank operates is desirable, and furnishes an automatic check upon the frequency of the discharge. Such a counter may be carried by a float suspended from the top of the tank, and operated by a ratchet, as in the one manufactured by the Pacific Flush Tank Company.

A standard flush tank was designed in Winnipeg under the direction of Col. H. N. Ruttan, City Engineer, which is vented by a pipe as shown in Fig. 227. This is one of the simplest forms of such apparatus. An entirely different type of flush tank is the Van Vranken (Fig. 228), the illustration showing the structures built in Concord, Mass. The sewer to be flushed ends in a well in the floor of the tank, which has a water-

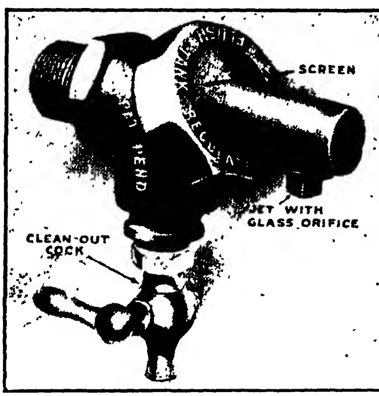


FIG. 226.—Flush-tank regulator. (*Pacific Flush Tank Co.*)

tight metal cover. A 5-in. siphon has its long leg carried down through the plate. The bottom of the leg is trapped in a tilting tray, which is so balanced that, when nearly full, its center of gravity is brought forward and it tilts down, allowing a part of its contents to flow out. This changes suddenly the air pressure in the siphon and starts the apparatus in action.

There are objections to any automatic siphon having moving parts, and the Van Vranken is no longer manufactured; but a number are still in service.

Another type of flush tank shown in Fig. 229 has been tested by W. T. Knowlton at Los Angeles and described by him.[1] This tank discharges through an 8-in. opening in its bottom which is normally closed by a hollow cast-iron valve with a rubber gasket contacting the cast-brass valve seat. The weight of the valve keeps it closed while the tank is filling and also supports a tilting basin swung from the opposite end of a walking beam. When the water surface reaches the desired elevation,

[1] *Munic. Jour. and Pub. Works*, 1919; **46**, 247.

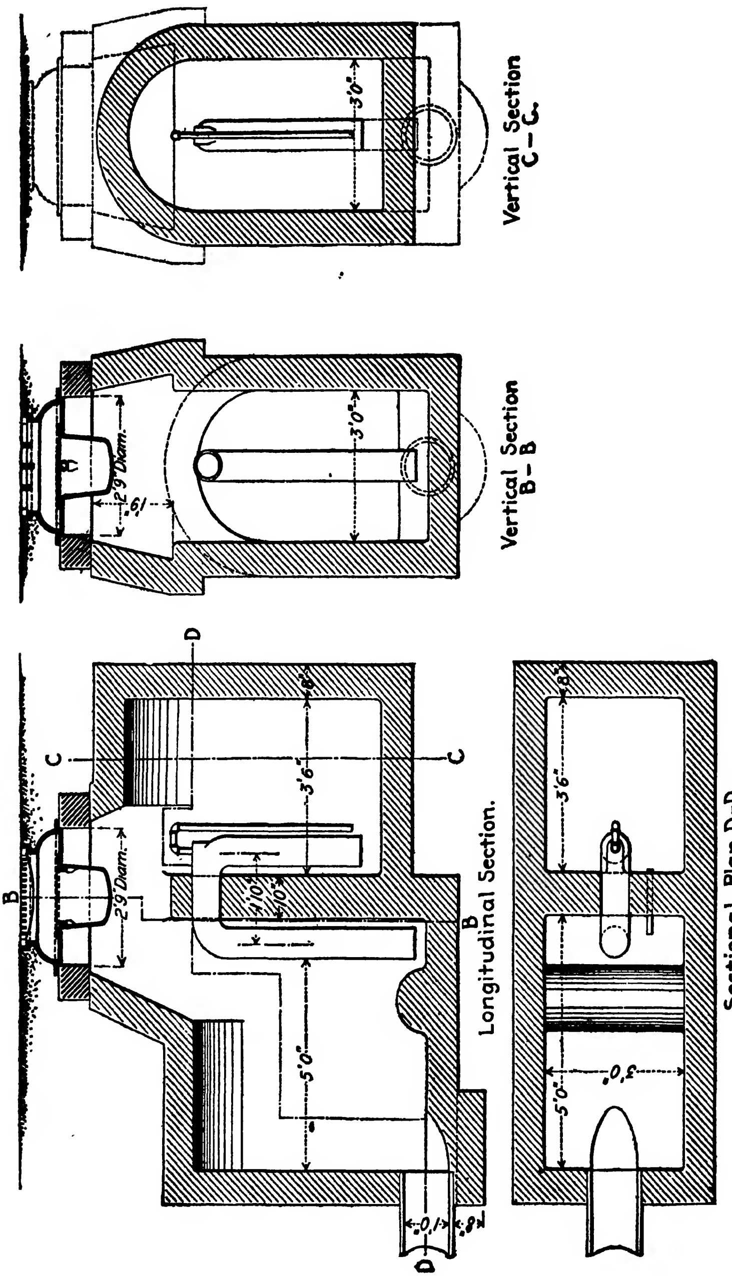


FIG. 227.—Standard flush-tank, Winnipeg.

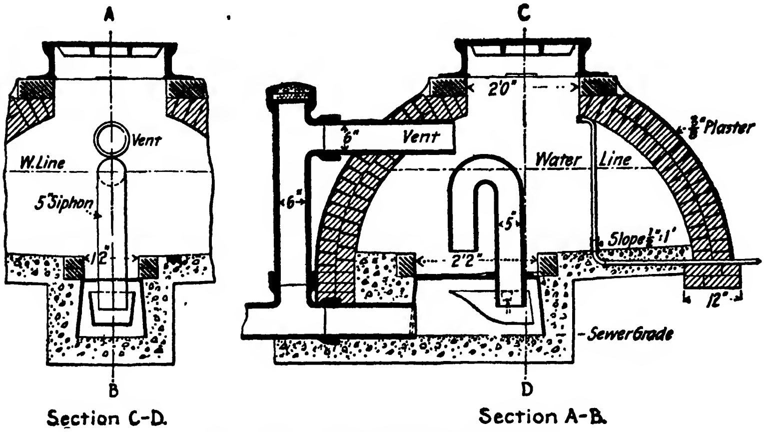


FIG. 228.—Van Vranken flush-tank.

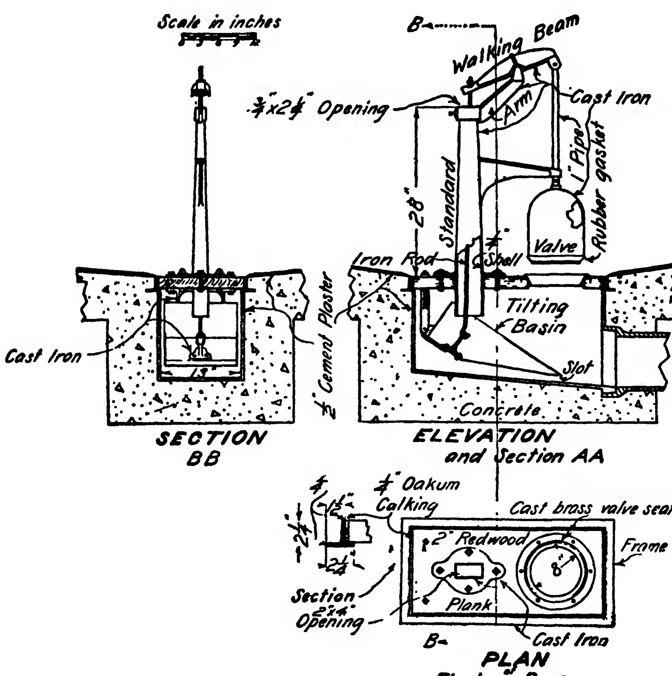


FIG. 229.—Los Angeles flush tank.

it overflows through a pipe into the tilting basin below. This basin, when filled, moves downward, thus lifting the valve and starting the tank discharging. The hollow valve floats until the tank is empty, when it again seats itself in a way similar to the operation of a flush tank of a water closet. The advantage claimed for this tank is that it discharged at rates of about 14 gal. per second, or from two to three and one-half times the discharge rate of siphons of equal diameter. The cost of this flush tank was $30 more than those formerly used by Los Angeles.

Value of Flushing.—The only theoretical analysis of flushing with which the authors are acquainted was presented by Asa E. Phillips, superintendent of the sewer department of the District of Columbia, in a paper before the American Society for Municipal Improvements in 1898. This paper gives the results of many measurements of the extent of flushing action in pipe sewers, and presents the following general discussion of the subject:

The object of the flush is to secure a periodic velocity of more than 2½ ft. per second in the upper portion of the sewer and to maintain the same to a point where the ordinary flow attains this rate. Disregarding the amount of normal flow in the sewer, it is evident that the quantity of water to satisfy this condition is a function of the diameter of the sewer and of its gradient. From the general consideration of the well-known formula for velocity, $v = C\sqrt{RS}$, remembering that for circular conduits the hydraulic radius is a direct function of the diameter, we may consider (1) the quantity Q varies directly as the square root of the radius and inversely as the square root of the slope, and, to complete the statement of controlling conditions, (2) that it varies directly as the length from the dead end to the point where the normal flow becomes sufficient to maintain a velocity of 2½ ft. per second. Under these assumed conditions, designating this distance by L, letting c represent the necessary modifying coefficient, the formula would take the shape, $Q = L\sqrt{R} \div c\sqrt{S}$.

Solving this equation for the data given on the Park Street line (given in the opening of the paper) we obtain a rough approximate for c of 190.

Let us now consider the factors which establish the value of L. If we let A represent the area of the cross-section of normal flow for any given gradient required to produce the velocity of 2½ ft. per second, and let D equal the increment in discharge for each linear foot of sewer in cubic feet per second, then $L = 2.5A \div D$, in which A is definitely determined by an application of Kutter's formula. The quantity D is evidently a function of the number of persons or premises tributary to the sewer and of their *per diem* water consumption. But these are variable quantities, rarely the same for two sewers. A uniform contributing population of 30 persons per 100 ft. of sewer with a daily flow per capita of 100 gal., three-fourths assumed to run off in 6 hours, would give a value for D of 0.00015 cu. ft. per second. Table 165 gives the value of A for different grades, and the corresponding depth of flow in inches. This table indicates the very small flow on the larger grades necessary to maintain a self-cleansing velocity, and the relation between the ordinary discharge and the grade within the limits given.

Table 166 gives the various quantities of water given by the formula for the foregoing grades and sizes under the conditions which have been stated, allowing an increased rate on long lines and a diminishing rate of flow on short lines from the average value of 0.00015 cu. ft. per second.

These results indicate that a very considerable modification of the volume of water should be allowed for lines of different gradient, and that the required volume diminishes very rapidly with an increase of grade; also that it is affected to a smaller extent by the size of the sewer, that for all sizes no flush tanks are probably required on slopes exceeding 2 per cent, and it may be inferred in such cases, also, that flushing at less frequent intervals is needed than the 24- to 48-hour discharge.

TABLE 165.—AREAS AND DEPTHS OF FLOW REQUIRED TO PRODUCE A VELOCITY OF 2.5 FT. PER SECOND IN SEWERS (*Phillips*)

Grade, per cent	Diameter of sewer							
	6 in.		8 in.		10 in.		12 in.	
	Area, square feet	Depth, inches	Area, square feet	Depth, inches	Area, square feet	Depth, inches	Area, square feet	Depth, inches
0.5	.....	...	0.229	5.0	0.226	4.3	0.237	4.1
0.75	0.125	3.9	0.130	3.2	0.137	3.0	0.150	2.9
1	0.095	2.9	0.101	2.7	0.108	2.5	0.115	2.4
2	0.043	1.7	0.050	1.6	0.055	1.6	0.060	1.5
3	0.031	1.3	0.035	1.3	0.037	1.2	0.041	1.2
4	0.022	1.0	0.025	1.0	0.028	1.0	0.031	1.0
5	0.017	0.9	0.021	0.9	0.024	0.9	0.027	0.9

TABLE 166.—GALLONS OF WATER REQUIRED FOR FLUSHING (*Phillips*)

Grade, per cent	Diameter of sewer		
	8 in.	10 in.	12 in.
0.5	80	90	100
0.75	55	65	80
1	45	55	70
2	20	30	35
3	15	20	24
4	10	15	20
5	8	10	15

An investigation of the action of water in flushing sewers was made by Prof. H. N. Ogden[1] at Ithaca, N. Y., about 1898. This investigation was begun to determine the necessity of a flush tank at the end of every lateral sewer in that city, in accordance with a recommendation made by the designer of the system. Ogden's correspondence with other engineers showed a wide diversity of opinion, some preferring hand flushing, others automatic flushing, and still others combinations of the two. A few had taken up hand flushing because of disastrous experience with automatic apparatus, and a few had adopted flush tanks because they found it impracticable to obtain good hand flushing.[2] Little practical information apparently was obtained, although one engineer reported that experience on the sewer system under his charge indicated that one flush daily on a 2 per cent grade was as effective as two flushes daily on a 0.5 per cent grade, each flush being of 300 gal. The general opinion was that occasional flushing was needed on the upper ends of all laterals on grades below 1 per cent.

Ogden's experiments were made on 8-in. pipe sewers, each with a 4-ft. manhole at its upper end. The end of the sewer was stopped with a pine board having a 5-in. orifice, closed by a rubber-faced cover. The manhole was filled with water to depths of 4 to 6 ft. and when the cover was removed the water was discharged at rates of 0.89 to 1.1 cu. ft. per second. The depth of this discharge and its effect in moving gravel were observed at successive manholes down the sewer. Flushes of 20, 30, 40, 50, and 60 cu. ft. were used successively.

As a result of these investigations, Ogden reached the conclusion that the volume of water discharged should not be less than 40 cu. ft., and the effect of the flush can hardly be expected to reach more than 600 or 800 ft. If tanks are used on grades greater than 1 per cent, 15 to 20 cu. ft. give as good results as larger amounts, but on such grades hand flushing will be more economical than automatic flushing.

In inquiries concerning the capacity of flush tanks a definite rule was received only from the Van Vranken Flush Tank Company, which stated that the capacity of the tank should be equal to half that of a length of sewer in which the grade produces a rise equal to the diameter of the pipe. It was the opinion of the manager of the Pacific Flush Tank

[1] *Trans. Am. Soc. C. E.*, 1898; **40**, 1.

[2] George G. Earl, superintendent of the New Orleans Sewerage and Water Works, informed the authors in 1913 that while there are automatic flush tanks on all dead ends of the sewers in that city, they are not operated constantly. "Instead, we have two men constantly going over the system, covering all flush tanks about twice a month and giving four or five flushes in rapid succession just as fast as a 1-in. pipe and gate valve, direct-connected to the flush tank, can fill them. This makes wave follow wave down the sewer, and we think saves water and gets better effect in flushing and reaches farther from the flush tank with an effective flush than two or three automatic discharges per day each. In addition to this we keep two gangs going over all sewers constantly with ball and flush cleaning."

Company that a flush of 175 gal. on a 1 per cent grade was sufficient, and on flatter grades twice that quantity of water should be used.

In the discussion of this paper, George W. Tillson stated that in Omaha on 6-in. lateral sewers with grades of ½ to 8 per cent and no flush tanks, a growth of fungus half filled the bore of the laterals in the course of a year or two. In later work of the same sort, flush tanks discharging every 12 hours were used at the dead ends of the laterals, and no trouble from the fungus was observed in such cases.

CHAPTER XVIII

REGULATORS, OVERFLOWS, OUTLETS, TIDE GATES AND VENTILATION

One function of a sewage-flow regulator is to prevent the surcharge of an intercepting sewer by closing an automatic gate upon the branch sewer connection, thus cutting off the sewage and forcing it to flow to another outfall. Another function is to regulate the flow in time of storm so that flow from one sewer carrying a heavily polluted storm flow may be admitted to the intercepter in greater proportion than from another sewer carrying a more dilute storm flow.

A storm overflow is designed to allow some of the excess sewage above a definite quantity to escape from the sewer in which it is flowing, through an opening.

The purpose of both devices is substantially the same, namely, to allow the ordinary flow of sewage to be delivered to a distant point of discharge, while causing the excess storm flow, which is very much less foul, to be discharged into the nearest watercourse. Sometimes regulators are used in combination with storm overflows to safeguard an intercepting sewer by entirely cutting off the sewage entering the intercepter when the latter is filled to a certain depth; the overflow allows the escape of excess storm flow; the regulator finally causes the entire flow in the branch sewer, both sewage and storm water, to pass the overflow and be discharged into the nearest watercourse.

REGULATORS

A flow or discharge regulator usually consists of an automatic gate operated by a float which rises or falls with the fluctuating sewage surface. When the intercepter is filled to its capacity, the gate closes entirely and further discharge of sewage into the intercepter is cut off.

Probably the best-known type of shear-gate regulator is shown in Fig. 230; this was used on the connections between the Boston main sewers and the Metropolitan intercepting sewers. The structure consists, in brief, of an orifice in the combined sewer, a pipe connecting this orifice with the intercepting sewer, a regulating gate, a float to operate the gate automatically, and a telltale pipe through which the height of sewage in the intercepting sewer is communicated to the float chamber.

The orifice in the combined sewer is designed of sufficient capacity to allow the proper quantity of sewage to pass through it. In some cases it is necessary to provide either a low dam in the combined sewer at a point immediately below the orifice or a depression in the sewer opposite the orifice to assist in diverting the sewage. The pipe leading from the orifice may pass through the regulating chamber and thence to the intercepting sewer. The regulating gate seats against a cast-iron nozzle which forms the orifice in the combined sewer. This gate is carried on the end of a lever, to the other end of which is attached a large float which rises and falls in the float chamber with the rise and fall of sewage

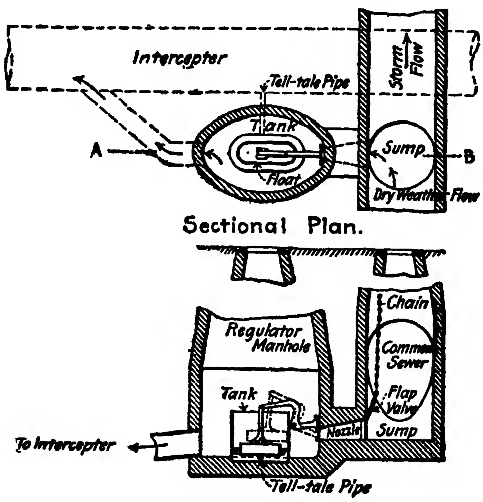


FIG. 230.—Old type of Boston regulator.

in the intercepting sewer, the communication of the height of sewage between the intercepting sewer and the float chamber being accomplished by means of a telltale pipe of small size which connects the two. Thus, as the depth of sewage in the intercepting sewer increases in time of storm, the float is raised and, correspondingly, the gate is lowered or closed. When the intercepter is as nearly full as desired, the gate through which the sewage flows is closed, thus preventing the flow of more sewage into the intercepter, and at the same time causing the sewage and storm water to flow past the orifice through the lower part of the original combined sewer into a river or tide water.

The experience with the mechanical features of these regulators has been satisfactory except in two respects. There has been a tendency in some installations toward the formation of deposits around the central

float chamber, while in other installations the valve has a tendency to become tightly jammed in the closed position by rags, paper, and matches caught between the valve and the valve seat. To overcome these objections, a later arrangement (Fig. 231) was developed by R. J. McNulty, Mechanical Engineer in the Boston Sewer Service. This regulator, which is patented, is made in sizes of 8 to 60 in. and is designed so that the floats may be actuated by the level of the sewage in either the

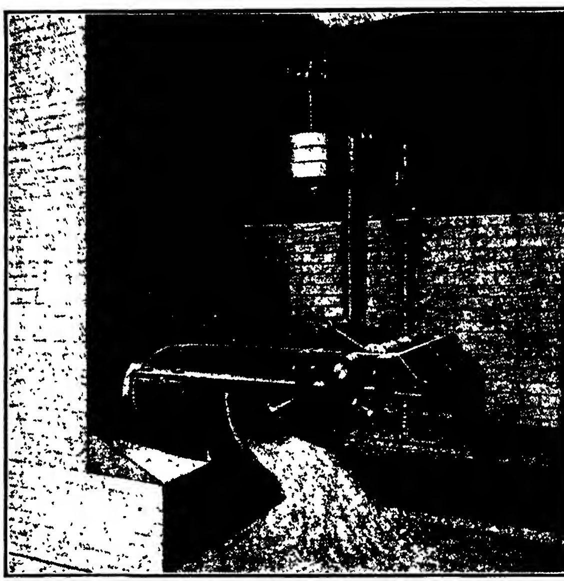


FIG. 231.—McNulty toggle-joint regulator.

combined sewer or the intercepter. It is manufactured by the McNulty Engineering Company of Boston.

Where it is desired to intercept only a constant volume of sewage, recourse may be had to a constant-flow regulator, of the type shown in Fig. 232. The depth of the sewage over the entrance to the vertical telescopic outlet pipe is maintained constant by lifting or lowering the pipe as the level of the sewage fluctuates. This motion is produced by the two large brass floats attached to the top of the pipe.

The simplest type of shear-valve regulator is shown in Fig. 233, and is made by the Coffin Valve Company, of Bostom. It has a cast-iron body which is bolted to the end of the branch sewer and projects into the intercepting sewer or a tank connected with it in which the sewage will rise to the same height as in the intercepter. The valve and frame are fitted with composition facings, hammered into dovetailed grooves

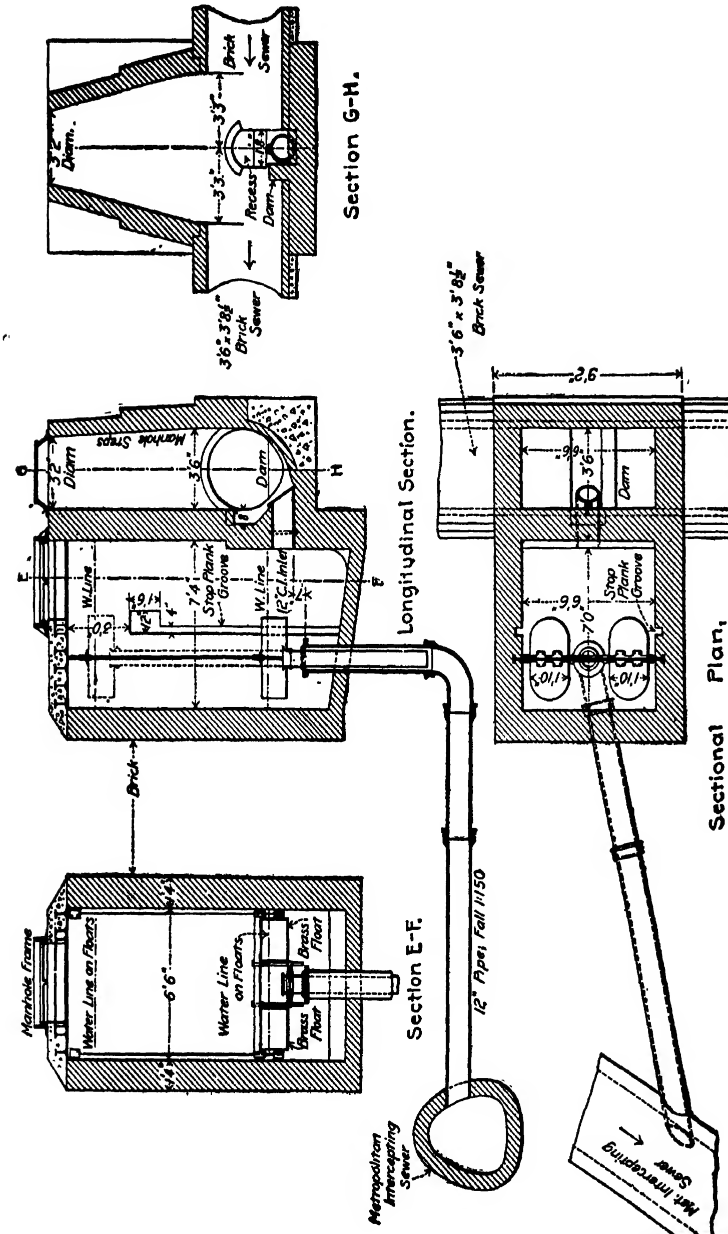


Fig. 232.—Constant flow regulator, Boston.

and pinned. The valve and its seat are machined and then scraped by hand to give a reasonably tight circumferential bearing. The steel shaft carries an adjustable copper float and a weight by which the action of the device can be varied somewhat.

Other types of regulators used at Syracuse, N. Y., are shown in Figs. 234 and 235, which require no comment. There is a limit, of course, beyond which it is hardly wise to expect such apparatus to operate automatically, and it is not surprising that one of these regulating valves refused to work, according to the chief engineer and designer of the intercepting sewerage system, Glenn D. Holmes, after it had become clogged with a 2- by 10-in. plank 5 ft. long, a roller 6 in. in diameter and 4 ft. long used in moving buildings, a 2-ft. length of a similar roller, a

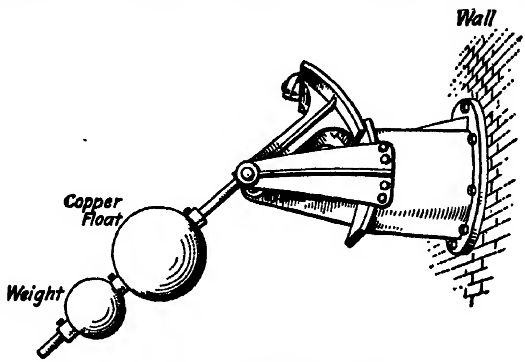


FIG. 233.—Coffin regulator.

4- by 8-in. timber 4 ft. long, mop and handle, broken crockery, rags, and small sticks. How such collections of large objects get into the sewers in the first place and are gathered at one spot after entering them, is one of the questions which occasionally puzzles the superintendent of any large sewerage system.

A type of regulator is used at Washington, D. C., in which the floats are operated by clean water from the city mains, admitted to the float chambers through valves controlled by the rise and fall of sewage back of an overfall dam. In 1913, Asa E. Phillips, then superintendent of sewers, stated that the most elaborate installation, shown in Figs. 236 and 237, had worked with absolute regularity for 2 years. It is so well balanced that it delivers the sewage from the combined sewer into the 3-ft. intercepter so long as the latter is not filled. As soon as the full capacity of the intercepter is being utilized, the regulator cuts off the flow to the intercepter, and as soon as the latter is able to receive more sewage, the regulator starts the flow again. The following is a description[1] of its operation:

[1] *Eng. Record*, 1912; 65, 312.

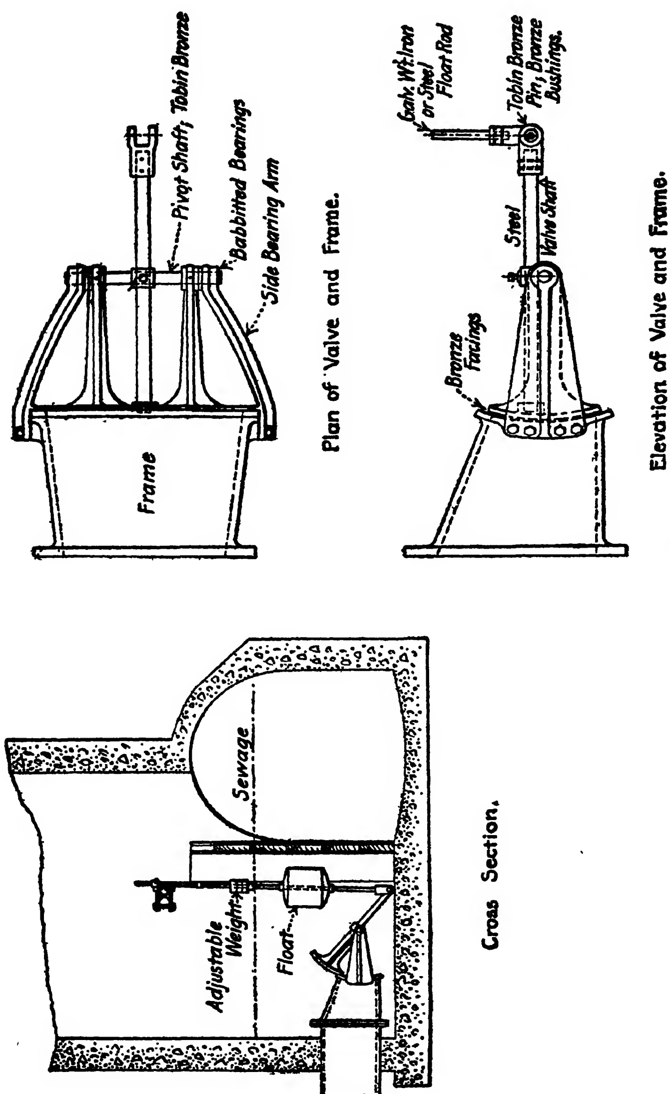


FIG. 234.—Regulator used at Syracuse, N. Y.

The apparatus for controlling the quantity of storm flow delivered to the 3-ft. intercepter, and for cutting out excessive storms, is located in an underground concrete gate chamber built just off the main line, and connected thereto by a 3-ft. conduit. Above this connection the trunk sewer is transformed in section from a circular to a cunette section, thus forming a collecting channel for the diversion of the flow to the gate chamber. This

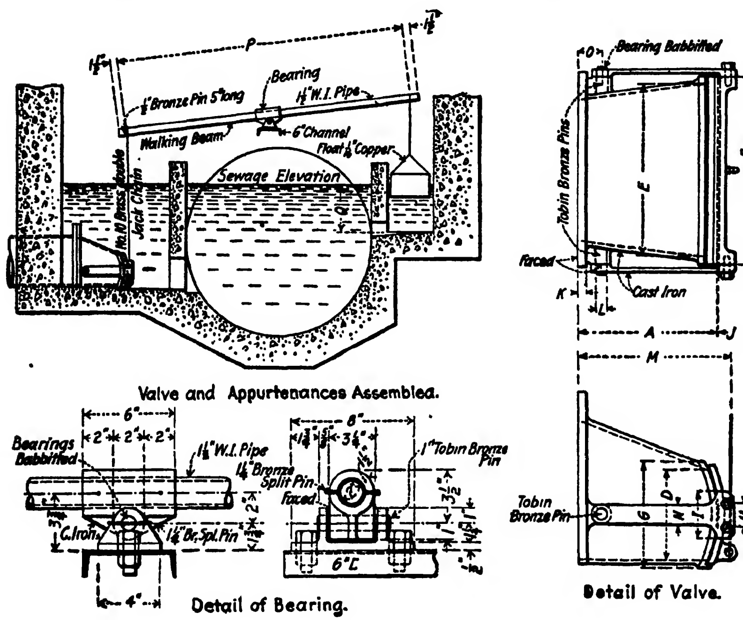


TABLE OF DIMENSIONS FOR VALVES

Size S	A	No.	B	C	D	E	F	G	H	I	J	K	L	M	N	O	Flange Bolts Diam.	Flange Bolts Length	Flange Bolts No.
8"	11¾"	10	13½"	11¾"	4¾"	11"	13"	6¾"	2¼"	4¾"	5/16"	¾"	1"	12¾"	2"	2¼"	¾"	2¾"	7
12"	14½"		19"	17"	7½"	15½"	17½"	9½"	2½"	5"	¼"	⅞"	1¼"	16⅛"	2¼"	2½"	⅞"	3¼"	10
18"	19¾"	2	25"	22⅝"	11⅝"	22"	24"	13⅝"	3"	6"	5/16"	1⅛"	1½"	21"	2½"	3"	1⅛"	4"	13
24"	24"		32"	29½"	16¼"	28"	30"	18¼"	5"	8"	5/16"	1¼"	1½"	25⅝"	2½"	3"	1¼"	4¼"	16

FIG. 235.—Regulator used at Syracuse, N. Y.

cunette extends as a tongue below the 3-ft. outlet conduit for the purpose of diverting from the intercepter the heavy material such as cobble and boulder, which excessive storms bring down from raw surface areas within this drainage district. Just where this tongue of the cunette dies out in the berm, a slight ridge is raised, forming a low cross dam for the purpose of holding the hydraulic gradient at such a level that the 3-ft. intercepter will run full before any discharge is spilled into the stream.

The automatic regulating apparatus is designed to entirely shut off the flow from the intercepting sewer just as soon as the latter is running full.

Under this condition, the flow in the trunk sewer is about level with the top of the diverting weir. This result is accomplished by means of two valves with disks in the form of cylindrical surfaces, which slide upon bronze seats in castings imbedded in a concrete bulkhead wall across the line of flow. From these disks arms project with floating balls of copper on the ends of

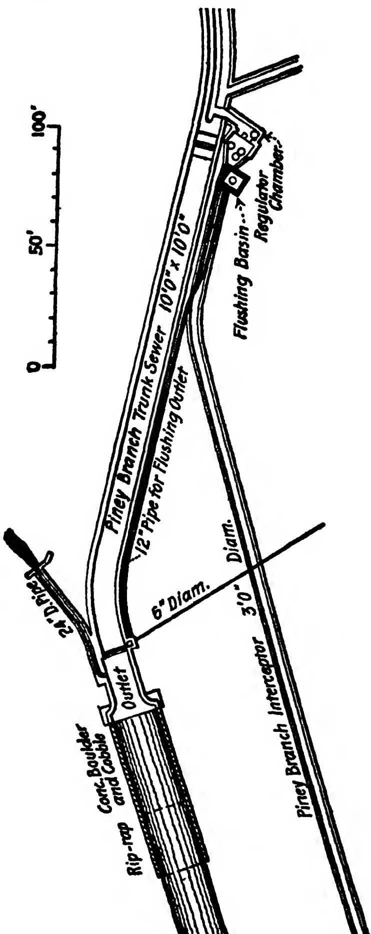


FIG. 236.—Plan of regulating works, Piney creek sewer, Washington, D. C.

same and working in a pair of concrete tanks, so that by automatically filling and emptying the tanks at the proper time the balls are made to rise and fall and to close and open the valves.

At the proper level below the diverting dam in the main sewer to give the required discharge, as checked by experiment in the shop, a small pipe is introduced and leads to a pair of outlets in the regulating chamber, each

one directly over a small funnel pail, hung from a lever arm in such a way that a downward movement of the funnel lifts a ball valve on a 2-in. pipe outlet from a 10-ft. capacity reservoir suspended from the roof of the gate chamber, which is filled through a float-controlled valve by a pipe connection with the city water main. This 2-in. pipe discharges directly into the float tank below on the floor of the gate chamber, and raises the large copper float which closes the automatic gate.

When the flow in the main sewer rises above the inlet pipe just below the level of the diverting weir, water passes into the two suspended funnels, filling them and thereby causing sufficient weight on the end of the lever arm to lift the ball valve on the outlet from the reservoir, which is connected to the float tank. The water rapidly rises in the latter, lifting the copper float and gradually closing the segmental slide-valve in the bulkhead wall. This shuts off the flow of sewage into the 3-ft. intercepter and automatically diverts same to Piney Branch. Peak load of the storm is thus entirely discharged into the stream. But as soon as the runoff is sufficiently reduced, the controlling gates open and the flow is once again diverted to the intercepting sewer. The operation is as follows:

When the flow in the main sewer drops to the capacity of the 3-ft. diameter intercepter it is just level with the inlet to the small pipes leading to the funnels, so that the flow which has kept the latter full is reduced below the discharge capacity of the funnel outlets, and the water therein quickly drains away, reducing the pull on the lever arm from which they are suspended, and thereby causing a counterweight on the extension of the arm to close the ball valve feeding the float tank.

These float tanks are drained by small outlet holes and when the feed supply is thus cut off they slowly empty and the floats descend, gradually opening the sliding valves, and delivering the discharge to the 3-ft. intercepter. This condition continues until the next excessive storm discharge. Meantime the reservoirs over the float tanks have filled from the city water supply, and are ready for the next storm.

During the period when gates are closed, the city water continues to flow into the reservoirs, and thence into the float tanks, thus keeping up the floats which hold the controlling gates shut, notwithstanding the small outlet holes which are always open and continue during this period to waste, the inflow, of course, being set to exceed this outflow. This is accomplished with a ½-in. supply pipe. The feed pipe leading from the main sewer to the funnels is protected by a screen and so connected and valved in the gate chamber that the city water pressure may be turned through same for flushing out the pipe and cleaning the screens. This connection also serves to permit the testing out of the apparatus at any time. Immediately after storms it is the practice to have an inspector visit the works to examine same and do any special flushing necessary.

Another type of installation in Washington is shown in Fig. 238. The regulators are for the purpose of shutting off the intercepting sewer completely at this place, when this becomes necessary, and diverting the sewage from the 6-ft. sewer leading to the pumping station over the sill

of a relief outlet, or into a bypass leading to a 6-ft. storm-water relief conduit running to the Potomac River. Below the elevation at which this regulating structure operates all storm water has to be pumped, as this is the lowest place from which there is a gravity discharge. The regulators are operated by the kind of apparatus described in connection with the first Washington installation. In November, 1913, Phillips wrote to the authors as follows:

We have 14 regulator chambers of this general character at present in the system, and some half-dozen additional planned for construction. All those in service have given most satisfactory results with the float-tank construction noted above. We have never attempted the hazardous experiment of placing the float directly in the sewer to be actuated by changes in level of the sewage flow itself.

In Rochester, N. Y., where sewers are built in tunnels as shown in Fig. 239, City Engineer E. A. Fisher adopted the type of regulator shown in that illustration. This has unusually sturdy members in proportion to the 12- by 20-in. opening which is under control, and is also unusual in that the disk is not designed to be able to shut off the discharge opening completely. This closing can be accomplished by hand, however. The operation of this regulator is described as follows in the report (1913) of Fisher on the sewage disposal system of Rochester:

It is contemplated taking into the intercepting sewer all of the sewage and two and one-half additional volumes in time of storm. The storm water in the outlet sewers in excess of this quantity will pass on and discharge into the river, the existing sewers thus becoming overflows beyond the point of interception. In order to control the flow to be diverted into the intercepter, chambers will be constructed in which regulating devices will be installed that will automatically maintain the required volume of discharge. These regulating devices will be operated by a float located in a chamber in which the water will rise and fall as the volume entering the chamber is in excess of, or less than, the volume discharging. As the water rises the float will operate a shutter closing the inlet, thereby reducing the volume entering until it is equal to the volume discharging; or if it grows less than the volume discharging, the water in the chamber will naturally fall, thereby causing the float to again open the shutter. The discharge from the chamber is fixed by the size of the opening and a given head. In each case the regulating device must be adjusted so that the float will begin to operate by closing the shutter when this given head is reached. In order to provide for a larger discharge, as the amount of sewage increases from year to year, the size of the opening from the chamber will be enlarged in order to give the area required with the given head to produce the discharge desired.

A special regulator has been constructed by George A. Carpenter, City Engineer of Pawtucket, R. I., using a gate valve operated by a hydraulic plunger, controlled by the old type of Venturi meter recording

apparatus, actuated by a float. In this case it was desirable to have the entire dry-weather flow and the first wash of the streets at times of storms taken to the treatment works, and to turn the entire flow of the sewer into the nearest watercourse when the dilution reached a certain point, reversing the operation when the total flow fell below another predetermined amount, less than that for which the gate was closed. The sewage flows through an orifice in the bottom of the diversion chamber into a pipe upon which the hydraulic valve is established. A float in the diversion chamber moves a vertical rod upon which are tappets, one of which controls the opening and the other the closing of the hydraulic valve. When the quantity reaches that for which the valve should be closed, the tappet trips the Venturi register apparatus, which thereupon operates a small valve admitting water from the city water mains to the hydraulic cylinder and closing the valve. When the flow again falls to the point at which the valve should be opened, the other tappet trips the mechanism to reverse the valves and open the main valve. Since the only power required from the mechanism is that consumed in opening and closing the small valves in the pressure pipes, it has been found that one winding of the weights of the Venturi recording apparatus is sufficient for more than 200 operations of the hydraulic valve.[1]

At Cleveland, Ohio, where regulating valves of the walking-beam type were tried unsuccessfully, the gate which was operated by the float was plane, rather than curved. The gate frame was made of cast iron and provided with a phosphor bronze seat; the gate was of cast iron. The main bearings of the walking beam had bronze bushings and attention had been paid in the design to the elimination of friction and opportunity for any binding of the parts. The following note on the failure of this regulator was furnished by J. M. Estep, Assistant Chief Engineer of the Department of Public Service of the city:

> The trouble with this type of regulator has been that the sliding gate, which shuts off the flow at a certain elevation of the storm water in the chamber, fails to operate properly in the phosphor-bronze slides, and I think the gate probably remains open so that this type of overflow acts just as the ordinary overflow where a diversion dam is used.

The construction of automatic regulators and the nature of the sewage and water passing through them are such that frequent inspection is necessary to assure their effective operation. Regulators and tide gates should be inspected every day, and immediately following storms the cleaning and inspecting force should be increased so that all regulators which have become clogged can be put into working condition as soon as possible. It is only by this means that automatic regulation will be satisfactory.

[1] A description of the valve will be found in *Jour. Boston Soc. C. E.*, 1914; 1, 149.

OVERFLOWS

Storm overflows may be of the following types; overfall or side weirs, siphons, baffled weirs, and leaping weirs.

Side Weirs.—An overfall weir is usually constructed in the side of a sewer, and the excess flow escapes over the crest when the elevation of the sewage is above that of the weir. One method of design of such a structure is described fully by W. C. Parmley,[1] in a paper on the Walworth Run sewer in Cleveland. His study was primarily a mathematical treatment of the problem as there were no experimental data upon which it could be based, and later experiments have not borne out his assumptions.

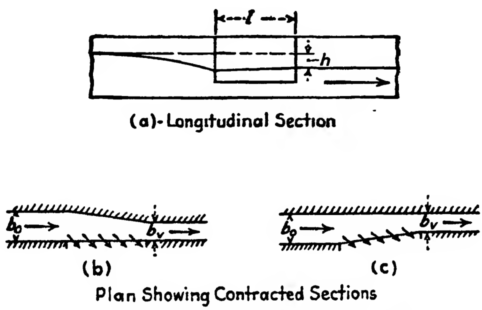


FIG. 240.—Discharge over side weirs.

The earliest experiments to determine the discharging capacity of side weirs that have come to the authors' attention, were those of Hubert Engels of Dresden. From his tests, conducted on a model, he derived the formula

$$Q = \tfrac{2}{3}\mu\sqrt{2g}\,\sqrt[3]{l^{2.5}h^{5.0}}$$

where Q = discharge over the side weir in cubic feet per second
l = length of weir crest in feet
h = head on weir in feet

Substituting 0.414 for $\tfrac{2}{3}\,\mu$ as determined by his tests, this expression becomes

$$Q = 3.32l^{0.83}h^{1.67}$$

The discharge may be increased by contracting the channel as shown in Fig. 240, *b* and *c*. In either case the ratio b_o/b_v does not appear in the formula which, for the case of contracted channels, becomes

$$Q = 3.32l^{0.9}h^{1.6}$$

The depth at the upper end of weir crest in Fig. 240*a* is seen to be less than at the lower end, the explanation given being that the increasing

[1] *Trans. Am. Soc. C. E.*, 1905; **55**, 341.

depths on the weir crest result from the conversion of kinetic into potential energy along its length.

This phenomenon is not reported by Coleman and Smith[1] nor by Prof. Harold E. Babbitt[2] of the University of Illinois in their experiments on similar weirs, though the former state that the water in the flume rises downstream from the weir due to a decrease in velocity with depth. It did, however, occur in the large structure described by W. H. R. Nimmo,[3] and was observed by Tyler, Carollo and Steyskal in experiments on side weirs at the Mass. Institute of Technology, 1928. Working with a model flume, 4¾ in. by 6 in. in section and with weirs 1½ to 24 in. long, Coleman and Smith summarize their tests in the following formulas:

$$l = 29.06b^{1.4}h^{0.513}$$
$$= 0.548bvh_1^{0.13}\left(\frac{1}{\sqrt{h_2}} - \frac{1}{\sqrt{h_1}}\right)$$
$$Q = 0.671l^{0.72}h_1^{1.645}$$
$$= 1.674bl^{0.72}h_1^{1.645}$$

b = width of flume
v = velocity of approach
h_1 = head on upper end of weir
h_2 = head on lower end of weir

(All units in feet and seconds)

Comparing the formulas for Q, it is apparent that the latter gives considerably smaller values than that derived by Engels. It should be remembered that Coleman and Smith's results were obtained from tests on a model and a proper conversion factor should apparently be applied in accordance with the law of hydraulic similitude before use for actual structures.

Babbitt experimented with weirs made by cutting out the sidewall of 18- and 24-in. vitrified pipe sewers, with crests from 16 to 42 in. long. His results are expressed in the formula

$$l = 2.3vD \log \frac{h_1}{h_2}$$

Where l = length of weir in feet
v = velocity of approach in feet per second
D = diameter of the sewer (circular) in feet
h_1, h_2 = head on upper and lower ends of the weir

This formula is in a better form for use than the Engels formula as it takes into account h_1 and h_2, the purpose of the weir being to reduce h_2 to a minimum.

[1] Coleman, George Stephen, and Dempster Smith, "The Discharging Capacities of Side Weirs," London, 1923.

[2] Babbitt, "Sewerage and Sewage Treatment," Second Edition, 112.

[3] "Side Spillways for Regulating Diversion Canals," *Trans. Am. Soc. C. E.*, 1928; **92**, 1561.

In Engels' experiments, which he described in "Mitteilungen aus dem Dresdener Flussbau-Laboratorium,"[1] discharges of from 0.52 to 6.35 cu. ft. per second in the main channel with 0.12 to 2.95 cu. ft. per second over the side weir were employed, the width of main channel varying from 0.67 to 6.56 ft., slope from 0.00,009,1 to 0.001, and length of weir crest from 1.64 to 32.8 ft.

Nimmo made one experiment upon a side weir about 180 ft. long in a timber flume at the head of the Ouse-Great Lake Canal, in Tasmania, which is described in the paper referred to above. The quantity of water discharged over the side weir, in a length of 172 ft., was 235 cu. ft. per second, out of 360 cu. ft. per second entering the flume, leaving 125 cu. ft. per second passing on down the canal. He developed formulas for computing the profile of the water surface opposite the side weir. The

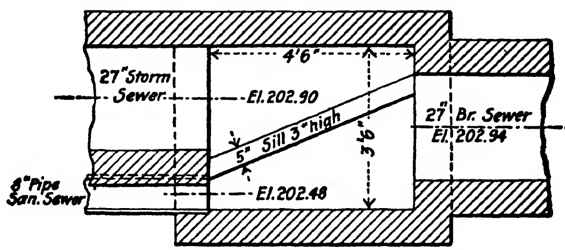


Fig. 241.—Overflow manhole, Cleveland.

computed water surface conformed very closely to that observed in the test. The water entered the flume through a short chute, and perhaps as a result of this, there was no flow over the weir at its upstream end. The crest of the weir was approximately parallel to the bed of the flume, with a total fall of about 0.5 ft. in the length of about 180 ft. The depth over the crest was approximately 1.2 ft. at the downstream end.

Nimmo's formulas are complex and do not permit of the direct computation of the quantity discharged over the side weir or of the length of weir required.

Oblique and Transverse Weirs.—The standard type of overflow structure used on the larger sewers in Cleveland, Ohio, is shown in Fig. 241. Attention is called to the fact that the fall for the dry-weather flow in this case is about 0.46 ft. in 4½ ft.

A marginal conduit has been built along the Boston shore of the Charles River to carry off the storm water from the area tributary to that river. This was necessary because of the construction of a dam across the river between Boston and Cambridge, converting a portion of the shore on either side into unusually attractive property facing a

[1] *Forschungsarbeiten auf dem Gebiete des Ingenieurwesens*, Papers 200 and 201; "Mitteilungen aus dem Dresdener Flussbau-Laboratorium, II," *Z. Ver. deut. Ing.*, 1920; 101–106.

fresh-water basin. The marginal conduit was designed to carry off the first storm wash, which contains most of the dirt from the streets and would pollute the water of the basin. As the district is closely built up, the area in question is practically impervious and after the first storm flow had carried off the dirt, it was thought that there would be relatively little more to be expected during the storm. The main conduit was provided, therefore, with a number of overflow chambers (Fig. 242), discharging the excess storm water into short outfall sewers leading to submerged outlets.

These overflow chambers were designed by E. C. Sherman under the direction of Hiram A. Miller. A curtain wall partially separates the overflow chamber from the conduit, so that sewage is drawn from the middle part of the stream and floating débris cannot be carried out into the fresh-water basin. The water in the basin is retained at El. 108 and

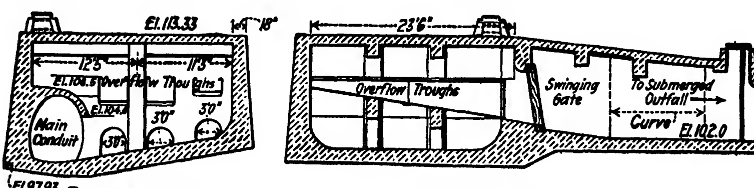


FIG. 242.—Overflow chamber, Boston marginal conduit.

as it was assumed that a loss of head of 0.5 ft. would be caused by the swinging check gate, the crests of the overflow troughs were placed at El. 108.5. The top of the conduit being at El. 106.2, the conduit is under a slight head at times when the overflow takes place. As soon as the troughs are filled, the head on the check gates causes them to swing open and permit flow into the basin to take place through the submerged outlets.

An overflow chamber of unusual arrangement was constructed in Boston about 1899 at a point where a brick sewer was crossed by a large brick conduit at a somewhat lower level, built in that year in order to carry the storm water from an area of about 650 acres, including a small brook known as Tenean Creek. This conduit was 9 ft. high and 10½ ft. wide at the crossing in question. The brick sewer was 3½ ft. high and 2⅔ ft. wide; where it crossed the conduit a reducer was constructed and a 36-in. pipe inserted in the arch of the conduit as shown in Fig. 243. The overflow channel starts from a chamber which is separated from the sewer by a dam and weir; at the outlet of this chamber there is a 3-ft. tide gate to prevent water in the drain from passing up into the sewer. Below this gate the overflow is 5 ft. wide by 4 ft. 4 in. high and enters the drain at an angle of 60 deg.

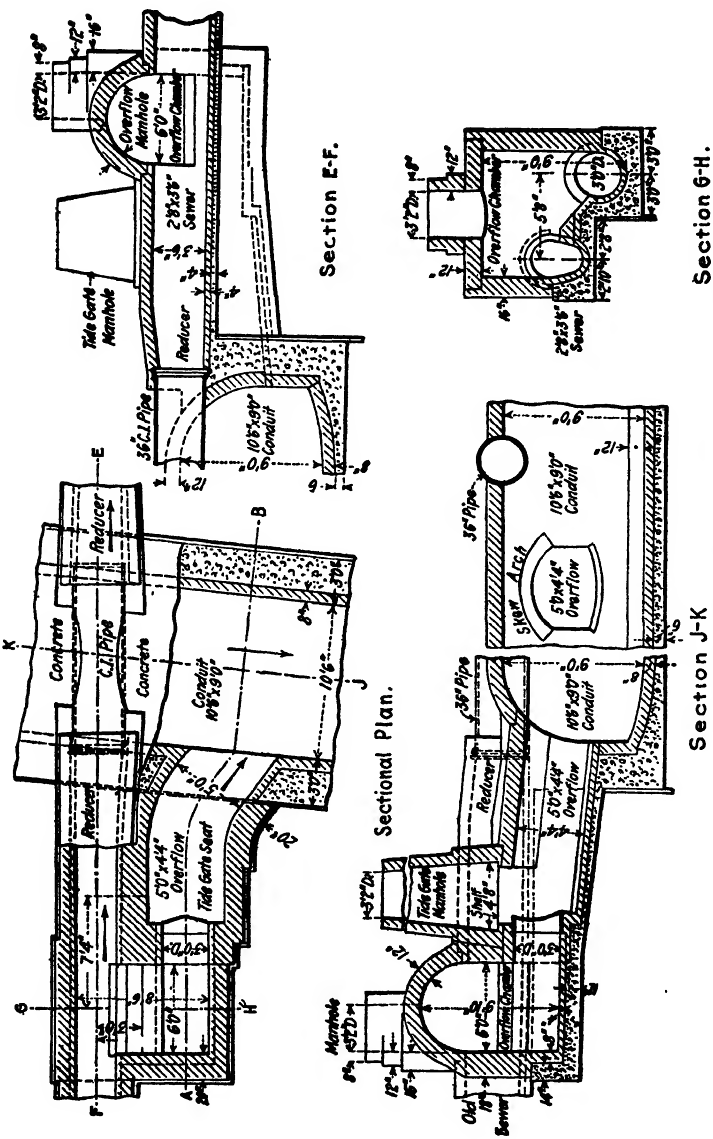


FIG. 243.—Overflow chamber and sewer crossing, Boston, Mass.

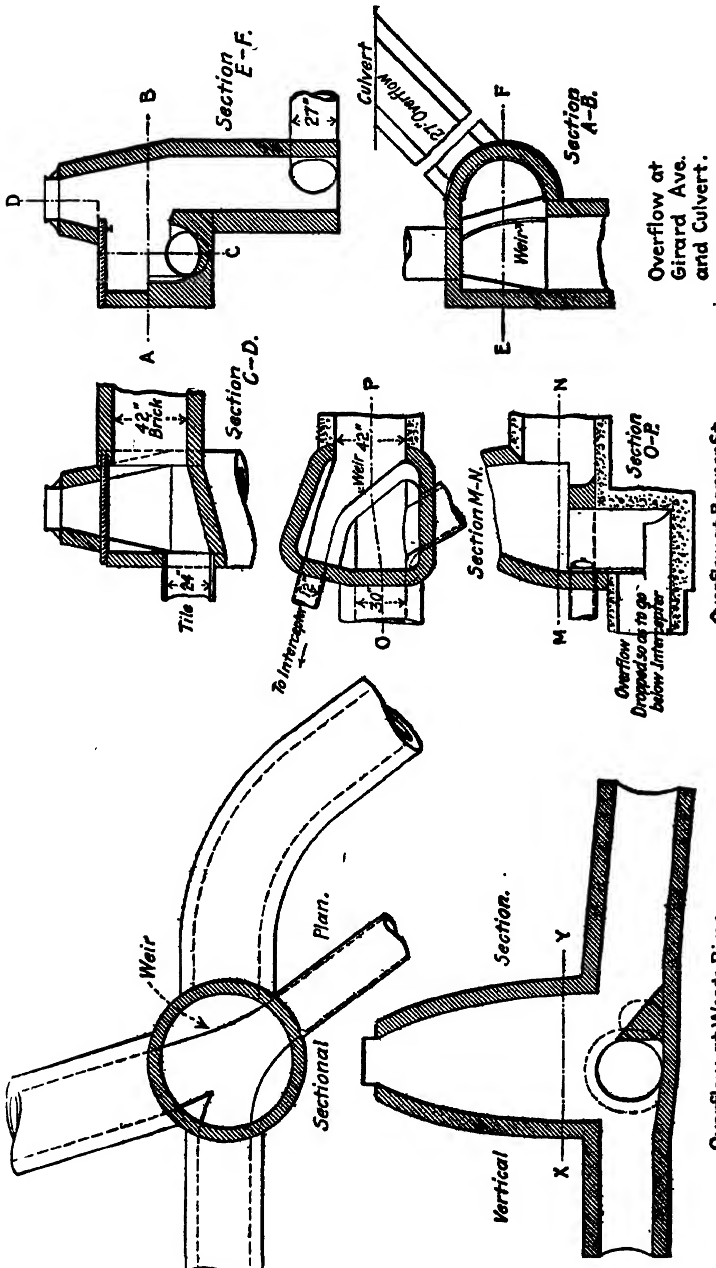


FIG. 244.—Overflow chambers used at Hartford, Conn.

At Hartford, Conn., referring to a district sewer system, Roscoe N. Clark, City Engineer, gave the following information:[1]

. . . A number of local sewers are brought together into a trunk sewer which is carried to a point near the intercepting sewer, from which one pipe, to carry the sewage flow, is built to the intercepter, and another, large enough for the storm water, is built to the river, brook, or storm-water cul-

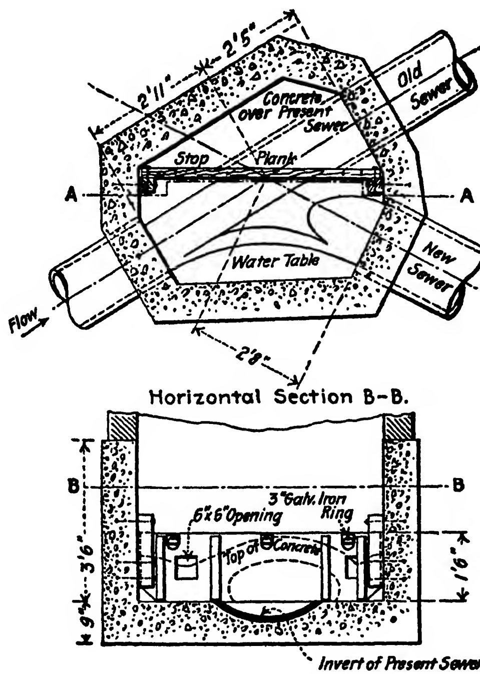


Fig. 245.—Stop-plank regulator, Hartford, Conn.

vert, as the case may be. In this case a weir is built across the overflow channel with its crest at the top of the sewage pipe, or above it, if it is desired to have the sewage pipe work under a head, as is sometimes done.

Examples of this are shown in the group of storm overflows illustrated in Fig. 244. The overflow at Bonner Street is a rather unusual one, because the overflow has been dropped to go below the intercepter; in most cases the intercepting sewers are the lowest at crossings of this kind.

[1] See also *Eng. News.-Record*, 1928; **100**, 402.

Where relief sewers must be built to take part of the sewage flowing in old sewers past certain points, use is made of weirs, as in the case of intercepting sewers. For example, in the case of the old Hartford sewer shown in Fig. 245, it was desired to remove practically all of the

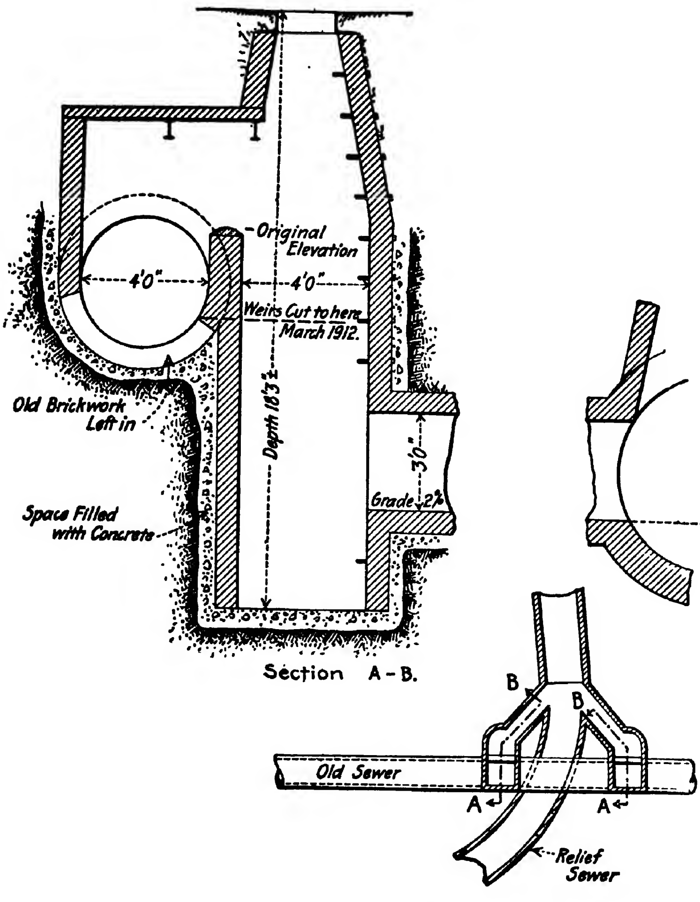


FIG. 246.—Twin overflow manholes on relief sewer, Hartford, Conn.

storm water but to keep the sewage in the old line. The latter was closed, except for about 3 in. next the invert, by an adjustable stop-plank which was expected to divert everything but the sewage into the new sewer. It was found in practice, however, that the height of 3 in. was not enough, and 6 in. would have been better to prevent the

opening becoming clogged. Another unusual Hartford connection between an old sewer and a relief sewer is shown in Fig. 246. There are two overflow manholes, and the crest of the weir in each, constructed about 1903, nearly at the top of the old sewer, was cut down after 10 years of service so as to lie only 1 ft. above the invert.

An overflow chamber at the end of a 30-in. cast-iron sewer at Tompkinsville, Staten Island, N. Y., is used for two purposes. The sewer is likely to operate under pressure at times, and consequently the sewage must have its velocity checked before it is discharged at the bulkhead

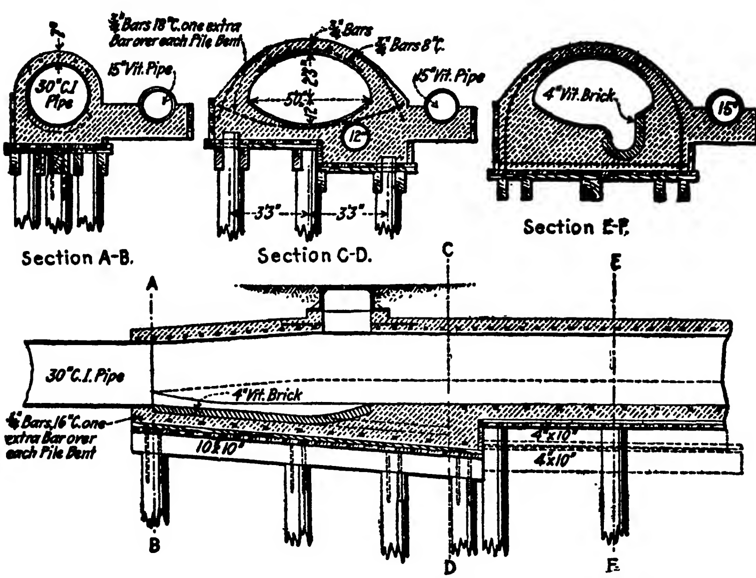


FIG. 247.—Overflow and transformer chamber, Staten Island, New York.

line of a pier 455 ft. long. To accomplish this a combined transformer and overflow chamber was built.[1] The transformer chamber (Fig. 247) is about 6 ft. long and is at the head of the overflow chamber, so-called, which is really what British engineers call a stilling chamber. It is 65 ft. long and its purpose is to reduce the velocity of the storm-water discharge by providing a greatly enlarged channel. This chamber and the 15-in. storm-water drain serving an adjoining railroad yard end at the bulkhead line, but the dry-weather flow is discharged through a 12-in. cast-iron pipe carried on slings under the pier floor to its outer end.

In the overflow structure in Fig. 248, one method of getting around the uncertainty and inefficiency involved in the use of side weirs is illustrated. In this design the weir is placed directly across the line

[1] *Eng. Record*, 1908; 57, 180.

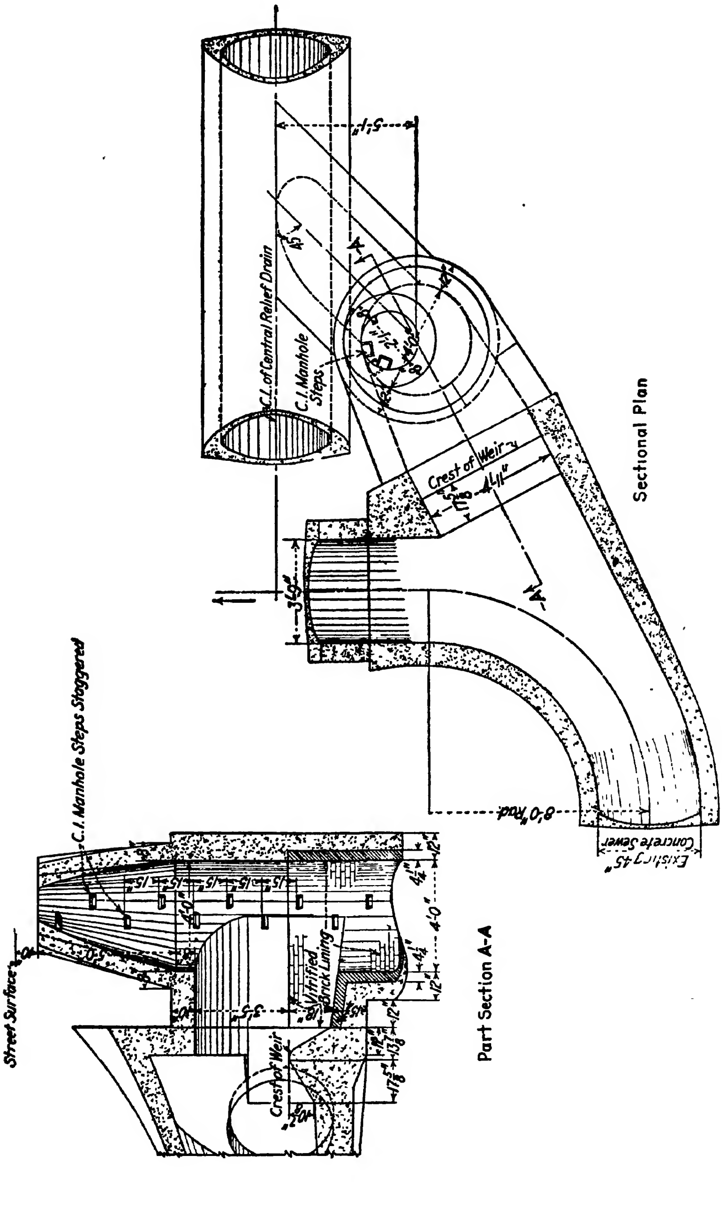


FIG. 248.—Overflow weir at Louisville, Ky.

of flow and the sewer is swung aside, thus reversing the arrangement used with side weirs. The discharge of a weir in this position can be determined by the usual weir formulas with greater certainty than can be done with the weir in the side wall of the conduit as the variation in head along the crest will presumably be smaller.

Baffled Weir.—A structure planned to avoid some of the uncertainties involved in the design of side weirs is the "baffled weir," illustrated in Fig. 249, which shows the Crouse Ave. connection to the intercepting

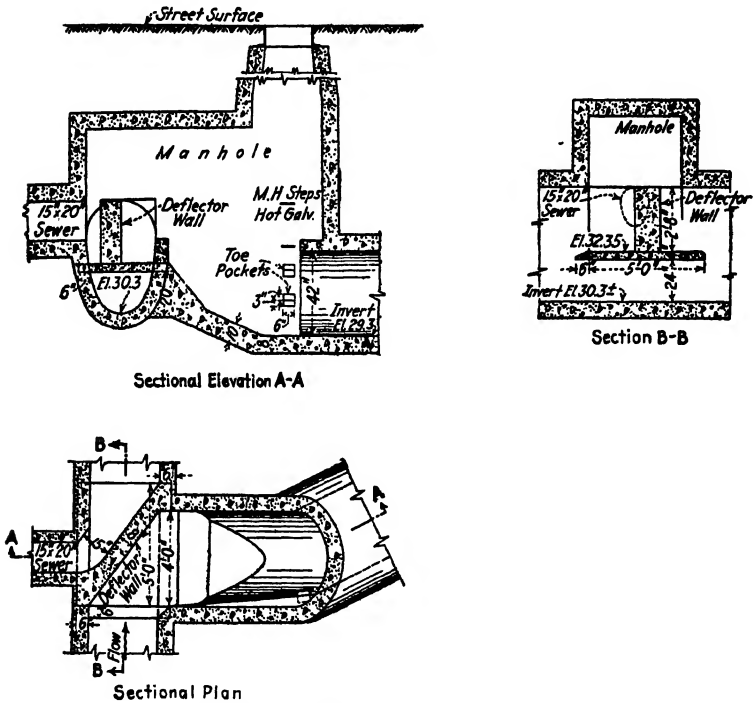


Fig. 249.—Baffled weir, Syracuse, N. Y.

sewer at Syracuse. This structure was built on the existing egg-shaped Crouse Ave. sewer by Glenn D. Holmes, Chief Engineer of the Syracuse Intercepting Sewer Board. A horizontal 4-in. concrete slab is placed across the sewer at an elevation which permits the desired maximum flow in the intercepter to pass underneath. A deflecting wall or baffle set at an angle of about 40 deg. with the axis of the sewer diverts the flow above this slab into a relief sewer depressed to allow for entry and velocity head losses. This structure is of further interest because it was built in an existing manhole under the main passenger tracks of

the New York Central Railroad, where construction work of any kind was difficult. Relief structures of this type have been used abroad and have the advantage over ordinary side weirs of economy in size of structure and more definite control of the quantity of sewage remaining in the sewer below the skimming plate.

A later modification that has been used by Holmes is to omit the horizontal cut-water slab and rely entirely on stop planks set in grooves in the sewer walls, to deflect the desired quantity of storm water. He used two 18-in. deflecting baffles set at an angle of 45 deg. with the direction of flow. The upstream baffle is 9 in. higher than the downstream baffle and comes into service at periods of maximum flow. The downstream baffle deflects ordinary excess flows and can overflow if it becomes clogged underneath.

Experiments at the Massachusetts Institute of Technology by Tyler, Carollo and Steyskal in May, 1928, indicated that the horizontal skimming plate was not advantageous, and also that a deflecting bulkhead placed squarely across the conduit, with its bottom edge at the elevation of the weir crest, was more efficient in deflecting excess flow over a side weir than one set at an angle. They suggested as a result of their experiments, that by making the elevation of the lower edge of this bulkhead adjustable, it could be set to pass the desired amount of sewage down the main conduit and would force much larger discharges over a side weir of given length of crest, than could be discharged by a similar weir without the bulkhead. This increase in discharge is due to an increase in head on the weir caused by the change of velocity head to static head above the bulkhead. A condition of non-uniform flow exists at and below the bulkhead and a jump may be produced. A bend or a flat gradient downstream may force the jump back against the bulkhead, which increases the discharge over the side weir, while complicating the hydraulic computations for the structure. While the range of the experiments was not great enough to prove that these results would be obtained under all conditions, the indications are significant.

In considering the hydraulic problems involved, the study must be largely theoretical, as there are few experimental data available upon which to base the design. If the area below the deflecting baffle be considered as an orifice, contraction will occur on the side in contact with the baffle. That this contraction may appreciably depress the surface of the sewage below and thus determine the elevation at which the baffle or skimmer should be set, is evident from experiments reported by Nimmo.[1] The increase in velocity due to pressure head should

[1] Nimmo, W. H. R., "Side Spillways for Regulating Diversion Canals," *Trans. Am. Soc. C. E.*, 1928; 92, 1561.

also be taken into account, together with the rise in level further downstream where the conditions of gravity flow in the sewer will have been

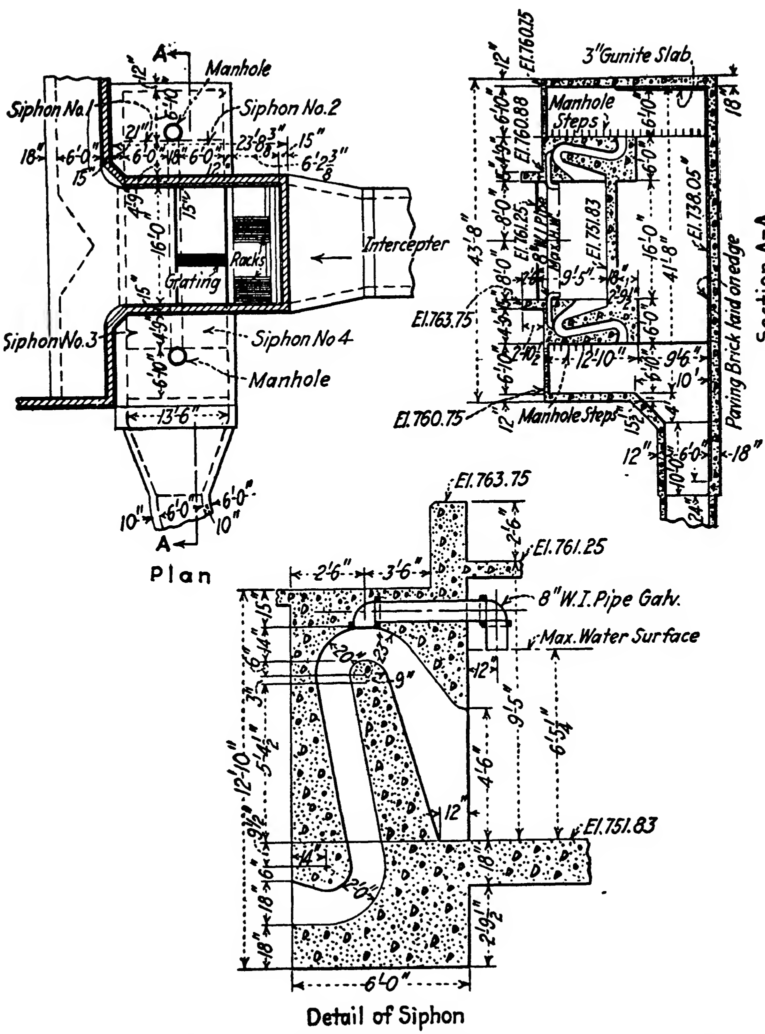


FIG. 250.—Siphon relief structure on intercepting sewer, Akron, Ohio.

reestablished. The elevation of the overflow weir and of the lower edge of the baffle may then be determined.

Siphon Spillways.—The siphon affords a means of regulating the maximum water-surface elevation in a sewer with smaller variations in

high-water level than can be secured with other devices. It works automatically and without mechanism and, as it utilizes all of the available head, discharges at higher velocities than do overflow weirs. The authors know of only two instances where siphons have been used for this purpose. While this device has obvious advantages, its infrequent use is due, doubtless, to inadequate information concerning certain matters pertaining both to its design and operation, such as, for example, the minimum head required for priming, the possibility of odors where vents are required to remove air from the outlet chamber, or possible noise and vibration from sudden starting and stopping of the siphon. Fig. 250 shows one of the siphon relief structures referred to, which was installed at the end of the intercepting sewer at Akron, Ohio. The purpose of this structure is to prevent the water level below the coarse racks from rising above the elevation at which the flow to the treatment plant is just equal to 93.6 million gallons daily, the design capacity of the plant, or, in case it becomes necessary to shut down the treatment plant, to remove the entire flow of the intercepting sewer. The sewer system of Akron is on the combined plan and the capacity of the intercepter is 252 million gallons daily. The siphon reliefs, four in number, are each designed for about 65 million gallons daily capacity. This structure was built in 1926 but has not been in service to date (1928).

The approximate cross-section for the siphon throat can be determined by the formula

$$Q = ca\sqrt{2gh}$$

Where Q = discharge in cubic feet per second
c = coefficient of discharge (0.6 to 0.8)
a = area of cross-section of throat in square feet
h = head in feet

A trial section may then be drawn, the losses determined for this section, and the corrected values of Q or a computed. The design then involves working out such details as the method of venting, shape of section to meet the particular requirements and the fixing of elevations of inlet, spillway, outlet, and air vent. An air vent with area equal to $a/24$ has been found to be ample. The siphon inlet should be large so that the velocity at entrance will be small, thus preventing large losses of head. The section tapers gradually to the throat and the lower leg, which may be either vertical or inclined, is of uniform section or slightly flared. The outlet is so constructed that air bubbles formed during priming will be carried out of the siphon.

When the water level rises to the spillway elevation water begins to flow in a thin sheet over the crest and falls against the opposite wall, sealing the siphon. This method of sealing is not necessary if the outlet is submerged. As the water level in the intercepting sewer rises, it

seals the air vent and the falling sheet of water carries out the air remaining in the siphon. This priming process is usually very rapid, requiring only a few seconds, so that the maximum water level corresponds approximately to that elevation at which the air vent is sealed or to the elevation of the spillway, if the latter is the higher. The siphon discharges

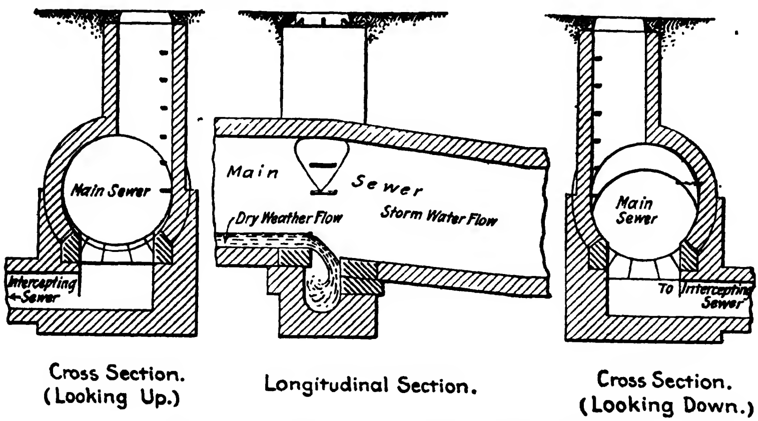


FIG. 251.—Leaping weir at Milwaukee, Wis.

till the water level drops below the vent and enough air is admitted to stop the siphonic action. The process is then repeated and the water level is held within the desired limits which are thus fixed by the relative heights of spillway and vent opening. By using siphons on both sides

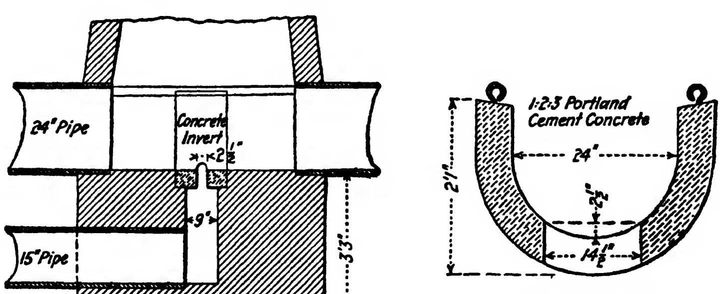


FIG. 252.—Leaping weir for pipe sewers, Cleveland, Ohio.

of the intercepting sewer the rate of discharge per unit length of relief chamber is increased and greater compactness of construction may be secured. The maximum operating head, theoretically, for a siphon is about 33.9 ft. at sea level, decreasing at the rate of about 1 ft. for 850 ft. increase in elevation above sea level. This is more than ample for

such uses as may be found for siphon reliefs on sewer construction. In a paper on "Siphon Spillways" by G. F. Stickney,[1] it is stated that the efficiency of siphons that have been constructed for reservoir spillways is from 60 to 70 per cent. Tests on a model at the hydraulic laboratory of the Massachusetts Institute of Technology gave a coefficient of dis-

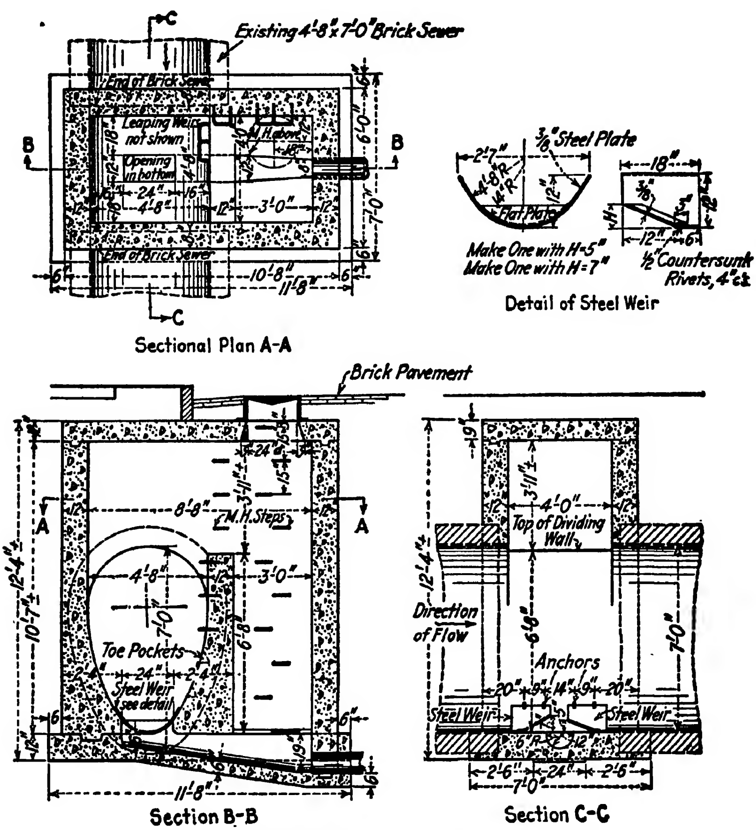


FIG. 253.—Leaping weir at Syracuse, N. Y.

charge of about 0.54, while Weirich (France, 1917–1918), also working with a model, obtained a coefficient of 0.97. The reason for the unusually high efficiency obtained by Weirich is not given in the published report and similar efficiencies have not been attained elsewhere.

A siphon relief structure has been constructed at the North Side Plant in Chicago, by the Sanitary District, but has not as yet (1928) been put into service.

[1] *Trans. Am. Soc. C. E.*, 1922; **85**, 1088.

Leaping weirs consist of openings in the inverts of sewers so constructed that the ordinary flow of sewage proper falls through the openings and passes to the intercepters. At times of storm, the increased velocity of flow causes most of the sewage to leap the openings and pass on down the sewers to the storm outlets. The first use of the device is commonly attributed to J. F. Bateman, designer of the first water works of Manchester, England.

The first use of the leaping weir in this country is believed to have been in Milwaukee, where 12 branches to the Menomonee intercepter were

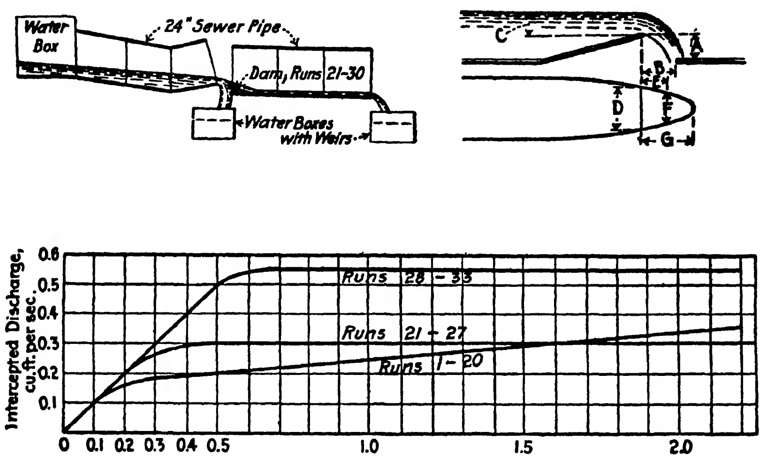


FIG. 254.—Syracuse leaping weir experiments.

connected by means of leaping weirs in 1887 and subsequent years. One of these connections is shown in Fig. 251.

The most simple type of leaping weir is that in which the dry-weather flow drops through a slit cut across the invert of the combined sewer. Such a weir used in Cleveland, Ohio, is shown in Fig. 252. The type is used on the smaller sewers and is known locally as the weeping weir; for larger sewers the manhole shown in Fig. 241 is preferred. Regarding the former Estep states:

> In the smaller systems this type is about as satisfactory as can be installed. We make calculations as to the amount of dry-weather flow in each case from the acreage, and then compute the size of the opening required to pass this amount of sewage.

In one branch of the intercepting sewer system of Syracuse, N. Y., leaping weirs were used at the connections of existing sewers with the intercepters, and float regulators were also employed to safeguard the

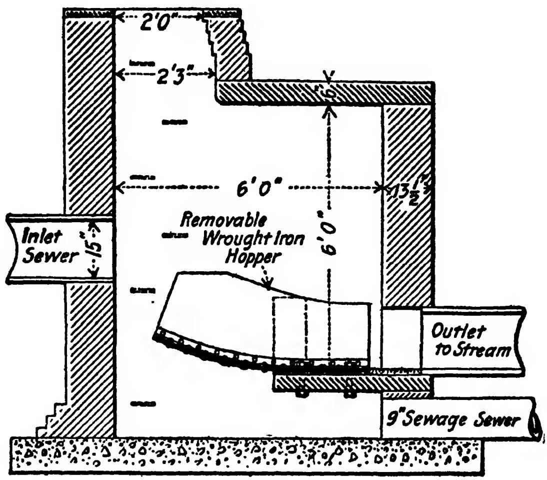
intercepters against surcharge. The later type of weir employed at Syracuse is shown in Fig. 253. It is formed with an inclined weir plate inclined upward so as to give a spouting effect and permit of a wider

TABLE 167.—SYRACUSE EXPERIMENTS WITH LEAPING WEIRS
(*Glenn D. Holmes*)

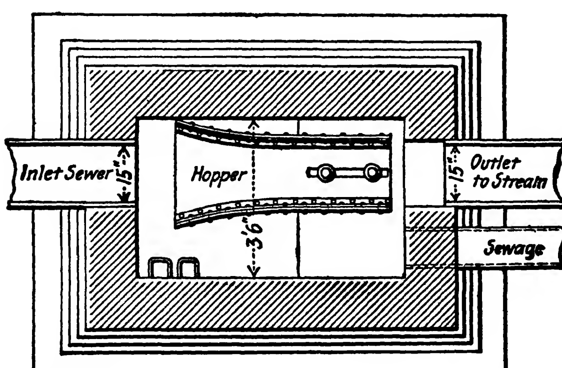
Run	Metered supply, c.f.s.	Inter-cepted water, c.f.s.	Disch. at outlet, c.f.s.	Dimensions, ft., Fig. 254						
				A	*B*	*C*	*D*	*E*	*F*	*G*
1	0.5	0.2	0.3							
2	0.15	0.15	0.0	0.4	0.4	0.17	1.08	0.21	0.75	0.4
3	0.26	0.2	0.07	0.4	0.4	0.20	1.21	0.25	1.17	0.4
4	0.36	0.21	0.21	0.4	0.4	0.24	1.27	0.25	1.00	0.4
5	0.70	0.22	0.48	0.4	0.4					
6	0.124	0.12	0.0	0.4	0.4	0.19	1.04	0.25	0.8	0.4
7	0.512	0.2	0.29(?)	0.4	0.4	0.29	1.35	0.25	1.2	0.4
8	0.438	0.18	0.12	0.4	0.4	0.25	1.25	0.25	1.05	0.4
9	0.407	0.19	0.19	0.4	0.4	0.27	1.3	0.25	1.12	0.4
10	0.708	0.19	0.5	0.4	0.4	0.34	1.5	0.25	1.3	0.4
11	0.862	0.25	0.62(?)	0.4	0.4	0.36	1.50	0.25	1.4	0.4
12	0.4	0.16	0.06	0.4	0.4	0.20	1.17	0.25	0.93	0.4
13	0.268	0.18	0.06	0.4	0.4	0.21	1.2	0.25	0.96	0.4
14	0.443	0.21	0.22(?)	0.4	0.4	0.27	1.3	0.25	1.10	0.4
15	0.731	0.22	0.40	0.4	0.4	0.31	1.4	0.25	1.25	0.4
16	0.758	0.22	0.52(?)	0.4	0.4	0.34	1.5	0.25	1.4	0.4
17	....	....	....	0.4	0.4	....	....	....	....	0.4
18	....	0.28	0.67	0.4	0.4	0.39	1.55	0.25	1.4	0.4
19	....	0.28	1.09	0.4	0.4	0.44	1.7	0.25	1.5	0.4
20	....	0.32	1.13	0.4	0.4	0.44	1.7	0.25	1.55	0.4
21	0.2′ dam	0.31	0.67							
22	0.2′ dam	0.31	1.05							
23	0.2′ dam	0.27	0.05							
24	0.2′ dam	0.29	0.11							
25	0.2′ dam	0.31	0.29							
26	0.2′ dam	0.31	0.93							
27	0.2′ dam	0.20	0.00							
28[1]	0.33	0.33	0.00							
29[1]	0.55	0.52	0.01	0.3	0.6	0.27	1.4	0.25	1.25	0.4
30[1]	....	0.52	0.05	0.3	0.6	0.30	1.45	0.25	1.2	0.4
31	....	0.55	0.20							
32	....	0.55	0.28							
33	....	0.55	0.69							

[1] Dam 0.2 ft. high in discharge pipe.

opening than in those originally built, thus reducing the probability of stoppage. During dry weather when the quantity of sewage is small and the velocity slight, the sewage drops over the weir into the channel lead-



Vertical Section.



Sectional Plan.

FIG. 255.—An adjustable leaping weir.

ing to the intercepter. At times of storm flow, the increased velocity causes the desired amount of the sewage to leap the opening, where it is caught behind a cast-iron inclined plate which may be adjusted in position, so as to vary the width of opening. Several of these weirs have been in use for over 15 years, and have given good satisfaction. Glenn

D. Holmes, Chief Engineer of the Syracuse Intercepting Sewer Board, states that 90 per cent of these leaping weirs have never received attention. When clogging occurs, notice is given by overflow into the water course during dry weather.

Tests were made upon a model weir of the type shown in Fig. 254 under Holmes' direction, the results of which are given in Table 167.

Another method of constructing a leaping weir with an adjustable width of opening, as suggested in Moore and Silcock's "Sanitary Engineering," is shown in Fig. 255. The following analytical treatment of the device is taken from that source, where it is credited to Prof. W. C. Unwin:

Let h be the head of water over the upper lip of the opening, x the horizontal distance from the upper lip to the edge of the lower lip on the farther side of the opening, y the vertical drop from the upper lip to the edge of the lower lip, and t the time for a particle of water to pass from one lip to the other. For practical purposes, the mean velocity of the water will be $v = 0.67\sqrt{(2gh)}$. But $y = 0.5gt^2$ and $x = 0.67t\sqrt{(2gh)}$. Then $y = 0.56x^2 \div h$.

From this the width which the jet will just pass over with a head h for any given difference of level can be computed. If, in addition, there is a velocity of approach, h must include the head necessary to give that velocity, *viz.*, $v^2/2g$.

Silt Chambers.—One of the objections to practically all diverting devices is the fact that silt is diverted into the intercepting sewers and is also likely to accumulate in the space reaching from the weir to the intercepting sewer. Not only the silt brought down by dry-weather flow, but that carried or rolled along the bottom by storm water, is accumulated in this space. This is likely to give rise to deposits. Even in so carefully designed an overflow as that at Cleveland, where there is no dead space behind the weir, the silt dragged along the bottom by storm water cannot pass the weir but must be carried on into the intercepting sewer.

In some cases, particular care has been given to the design of basins in which the silt carried down with the sewage can be retained and prevented from passing into the intercepting sewer. Two different ideas have been followed in designing such basins.

In one case a sump is constructed to retain the silt, forming practically a catchbasin from which the silt can be removed from time to time. An example is shown in Fig. 256, an illustration of an overflow and silt basin used at Harrisburg, Pa., built from the designs of James H. Fuertes. The drawing requires no explanation.

In the other type a depression is formed in the sewer, above the regulator, so shaped that the silt will be scoured out by storm flow and

carried down the storm sewer to the overflow. One of the best examples of this type is seen in the illustrations of the storm overflows at Washington, D.C. (Figs 237 and 238). In the first of these overflows the silt chamber consists of a depression in the invert of the main sewer. This

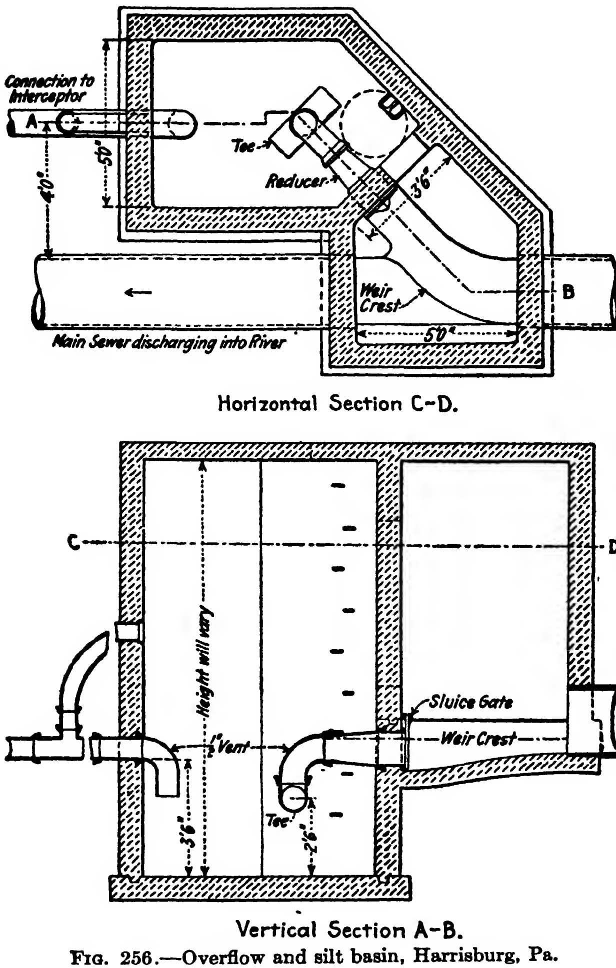


FIG. 256.—Overflow and silt basin, Harrisburg, Pa.

is sufficient to retain the silt brought down during ordinary times. At times of storm, when the regulator gate is closed, the high velocity scours out the accumulated silt and carries it over the dam to the storm-water outlet. In the second illustration there is a silt basin of considerable size in the chamber above the regulator gates. By opening a

sluice gate at the side of the chamber at times of storm flow, the silt can be forced into the storm sewer itself.

An objection to either of these designs is that an opportunity is afforded for organic matter to accumulate during low flows and to putrefy, thus forming offensive pools of sewage.

OUTLETS

Strictly speaking, the outlet of a sewerage system is the end of an outfall sewer at which the sewage is discharged. There may be a

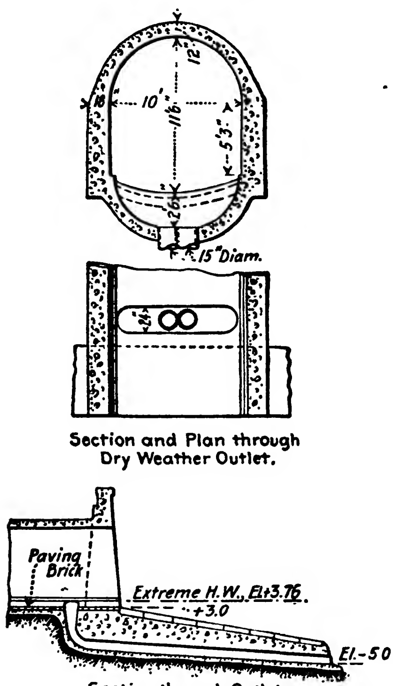


Fig. 257.—Dry-weather outlet, Minneapolis, Minn.

number of these outlets in case the city has several storm-water outfalls or overflows. In every case, the object should be to discharge the sewage at a point where its presence will cause no offense; the disposal of the storm water is not so difficult because it contains less organic matter and is not delivered continuously. Where the water is quiet the outlet of the outfall sewer is usually submerged to a considerable depth, while if the sewage is discharged into a stream flowing rapidly at all times, the outlet need not be submerged, provided the sewage

passes into the stream at a point where it is certain to be carried away and dispersed rapidly. In the case of outlets in tidal waters, it is generally impracticable to place them high enough so that they will not be entirely sealed at high tide; accordingly, the discharge of sewage is checked during the portion of the tidal flow when it is likely to be swept back along the shore, and accelerated when the tide is going out.

A different outlet is sometimes built for combined sewers than for those carrying nothing but sewage, because sewage must be discharged with much greater precautions to prevent nuisance than are required for the storm water flowing from combined sewers or drains. A combination of these conditions is illustrated in Fig. 257.[1] This outlet was built at Minneapolis, where the level of the Mississippi River, into which the sewage is discharged, fluctuates materially. The conditions made it practicable to build a double outlet, by which the dry-weather flow is carried out farther into the stream and to a lower level than the storm water. Two 15-in. cast-iron pipes run out below the paved apron in front of the storm-water outlet, and discharge the dry-weather sewage 5 ft. below low-water level in the river. The invert of the storm-water sewer is 9 in. below the high-water level in the river, so that the sewer will have a free discharge at all times.

Much the same plan is followed in the outlets of the sewerage system of Winnipeg, built from the plans of Col. H. N. Ruttan. The outfall sewers are built of concrete until they approach the banks of the rivers into which they discharge. Each outfall is then continued by a wooden sewer running out on pile bents at an elevation of 3 or 4 ft. above the river. Its outer end is closed by a large flap door, which floats upward when the river is in flood. About 10 ft. from the outlet end, a small pipe drops from the invert and is then carried forward on piles 50 ft. or more beyond the end of the main outlet, to take the dry-weather sewage well out into the stream in times of low water. These outlets are protected against the heavy ice floes by a sloping ice breaker of 6 by 6-in. timber, laid so as to carry the ice over the structures.

Where the sewage must be carried out into comparatively deep water, the outfall sewer is generally a cast-iron or steel pipe ending in a quarter bend or a tee, by which the sewage is discharged upward. A typical outlet of this character was built in 1913 to carry the effluent from the Rochester sewage treatment works into Lake Ontario. The pipe is 66 in. in diameter and made of half-inch plate, the straight portions being of the Lock-bar type with single riveted joints every 30 ft., and the bends of short sections with double riveted longitudinal seams. The submerged portion of the pipe was laid in a dredged trench 8 ft. deep until a depth of 35 ft. was reached, when the trench was shallower. The minimum backfill over the pipe was 2½ ft. The pipe terminates in a timber crib

[1] *Eng. Record*, 1911; **63**, 382.

7,000 ft. from the shore, and the discharge is at a point where the water is about 50 ft. deep. The crib or outlet structure is 46 ft. square by 24 ft. high, built of 12- by 12-in. hemlock timbers laid to form 25 pockets, which are filled with stone except where they are occupied by the pipe. The bottom of the crib is 3 ft. below the bottom of the lake and is surrounded with riprap extending 10 ft. up the sides of the structure. The top is 26 ft. below the mean low-water surface of the lake. The pipe discharges 10 ft. above the bottom of the lake, being raised as it passes through the crib. Built into the crib near the outlet is a three-way tee, the side openings being 38 in. in diameter. This was placed to provide

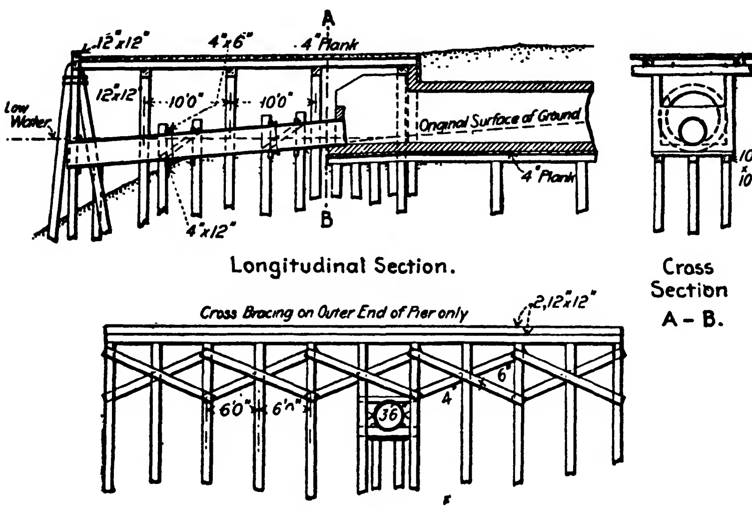


FIG. 258.—Outlet of joint trunk sewer, New Jersey.

for future extensions in case it is deemed necessary to discharge the effluent at more than one point, to promote more thorough dilution.

The outlet of the joint trunk sewer of northeastern New Jersey, on the shore of Staten Island Sound, is illustrated in Fig. 258. Like many of the outlets in the vicinity of New York Bay, it is below a wharf, which was constructed in this case in return for permission to establish the outlet at this place. The wharf is 60 ft. wide and about 40 ft. long, its base at the dock line making an angle of about 75 deg. with the axis of the sewer. The 72-in. brick sewer terminates in a brick chamber 7½ ft. square at the upper end of the wharf, from which a 36-in. cast-iron pipe extends a distance of 36 ft. to the bulkhead of the wharf, where its crown is 2½ ft. below low-tide elevation. This pipe is carried on piles independ-

ently of the wharf and is said to have enough capacity to discharge at the dock line, where there is a strong current, a volume of sewage equal to that delivered to the chamber by the 72-in. sewer. Owing to some doubt as to the stability of the foundations in this vicinity, the last 80 ft. of the brick sewer rests on a 4-in. plank floor 8 ft. wide, supported on three 10- by 10-in. stringers which are carried by three rows of piles. This structure was designed by Alexander Potter, Chief Engineer of the commission representing the seven communities interested jointly in the work.

The outlet of the southern outfall system of Louisville is shown in Fig. 259. It is at the end of a sewer 10 ft. 1½ in. high and 10 ft. 7½ in. wide. It includes a drop chamber 93 ft. long, built on concrete piles on the steep incline running down to an outlet structure 56 ft. long, the foundations of which rest on rock.

The crown of the outlet will be below the surface of the water in the river at all times after the proposed 9-ft. stage of the Ohio recommended by the War Department has been established by Congress. Before that time there may be occasions when the outlet will be partially exposed during extreme low water; during floods the river rises many feet above the outlet, the maximum being probably about 70 ft.

In determining the size of the drop and outlet structures, a hydraulic grade was assumed from the top of the sewer at the upper end of the drop chamber to the surface of the water in the river when at El. 415, or 32 ft. above the elevation for the 9-ft. river stage. This elevation is rarely exceeded during freshets in the winter; in June the height of the water has exceeded this stage only twice in 35 years, and remained above it for only a very short period of time then. Storms of great intensity are not frequent in this locality except in June, July, and August, and are very rare during the winter. The possibility of the occurrence of rainfalls of such high intensity as to tax the capacity of the sewer, occurring at a time when the river is above El. 415, was considered by the engineers in charge of the work to be very remote, and for this reason it was believed to be safe to base the design of the drop and outlet structures upon the hydraulic grade mentioned. The outlet structure will generally be submerged in the river, and occasionally at times of extreme floods the entire drop chamber and even the outfall sewer itself will be submerged for some distance. It was considered impossible to provide adequate drainage in the city during storms of great severity occurring at a time when the river is at an extreme flood stage. Such conditions are so rare that they must be construed as an "act of Providence," for which the city should not be expected to make provision.

There have been indications of a strong tendency of the river bank to move toward the river after the falling of the water in the late spring or summer. The bank is composed, to a large degree, of silt, which

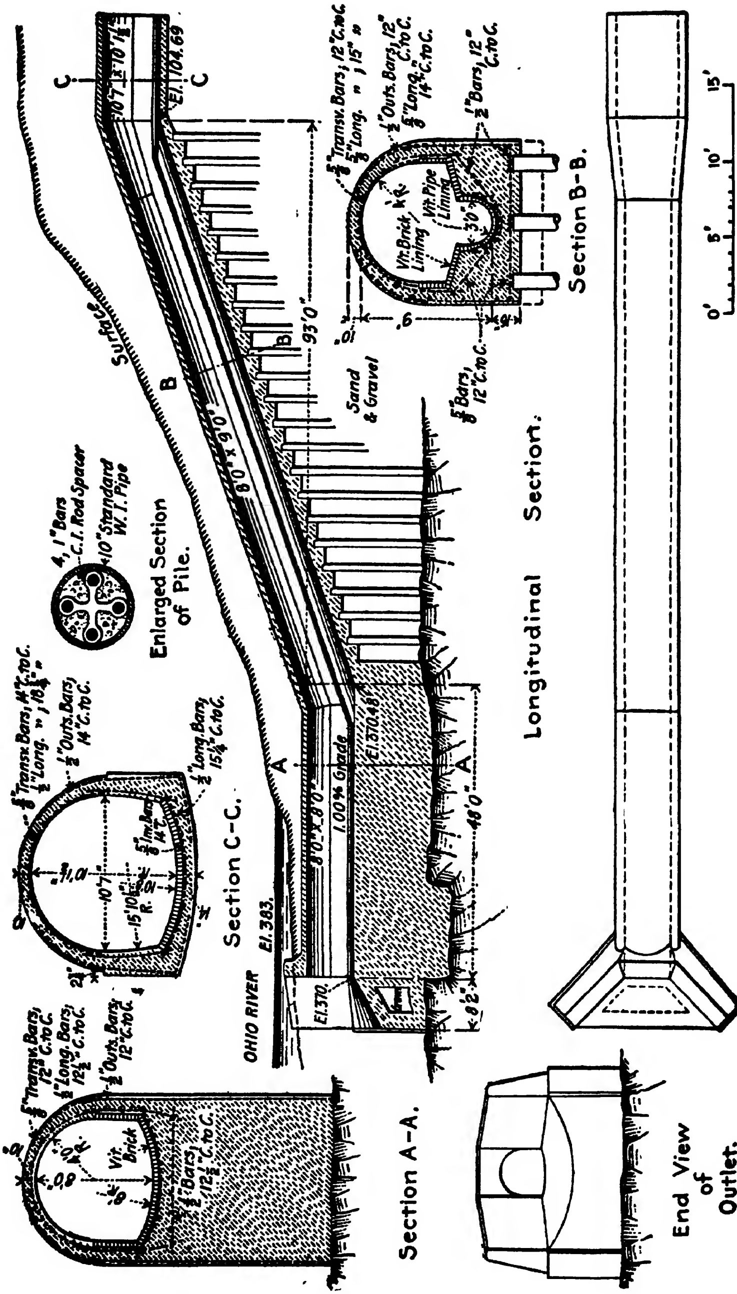


FIG. 259.—Southern outlet structure, Louisville, Ky.

becomes saturated during high stages of the river, and is very heavy when wet, possessing little stability. Underlying the silt is a bed of coarse sand and gravel, through which large quantities of water are flowing continually toward the river. The action of this water at the surface of the gravel probably tends to assist the sliding action of the silt above. In anticipation of any such action and its consequent effect upon the sewer at its outlet, the foundation was carried down to bed rock, as illustrated. For a short distance, the rock was excavated to a depth of 4 or 5 ft. and the foundation carried down in this pit to form a key to guard further against any movement.

The drop chamber was built on piles to assist in resisting any possible movement, as well as to support the structure in case, by any chance, it should be undermined by the action of the river. These piles extend to the rock where it is within 20 ft. below the masonry, and 20 ft. into the ground further up the bank, in all cases penetrating a long distance into the gravel underlying the silt.

The outlet structure is 8 ft. wide and 8 ft. high, with a semicircular arch, vertical side walls nearly 3 ft. high, and a comparatively flat but curved invert. At its outer end two wing walls were built out into the river, each making an angle of 45 deg. with the axis of the sewer.

The drop chamber has an arch, short side walls, and invert of the same dimensions as those of the outlet structure. In the center of the invert, however, there is a channel 3 ft. wide and 2 ft. 10 in. deep, lined with half-round vitrified sewer pipe. This channel is for the dry-weather flow, which will have a very high velocity. The pipe lining was used rather than vitrified brick, because of the absence of longitudinal joints, at which inverts on steep grades show the greatest amount of erosion, and for its good wearing qualities. On account of the velocity which will be obtained during the lower stages of the river, both the outlet and drop structures have been lined with vitrified brick to the top of the side walls.

The outlet of the northwestern sewer in Louisville is of the same general type but illustrates a different method of supporting such a structure. The cross-section in all places is 6 ft. 8 in. wide by 6 ft. high (Fig. 260). The outlet is submerged by the proposed 9 ft. stage in the Ohio River. For a distance of 77 ft. from the headwall the grade is 1 per cent. Then for a distance of 78 ft. the slope is 35.96 per cent, and finally for a distance of 256 ft., the slope is 5.88 per cent.

The mouth of the sewer is protected by head and wing walls of massive concrete, and the lower portion of the sewer rests on concrete columns built within steel caissons. The shell of the caisson was built of quarter-inch plate in 4- or 5-ft. widths, and each caisson was 5 ft. in diameter. These supports varied from 20 to 25 ft. in length. Each of them was surmounted by a cap which left 7 ft. of unsupported sewer

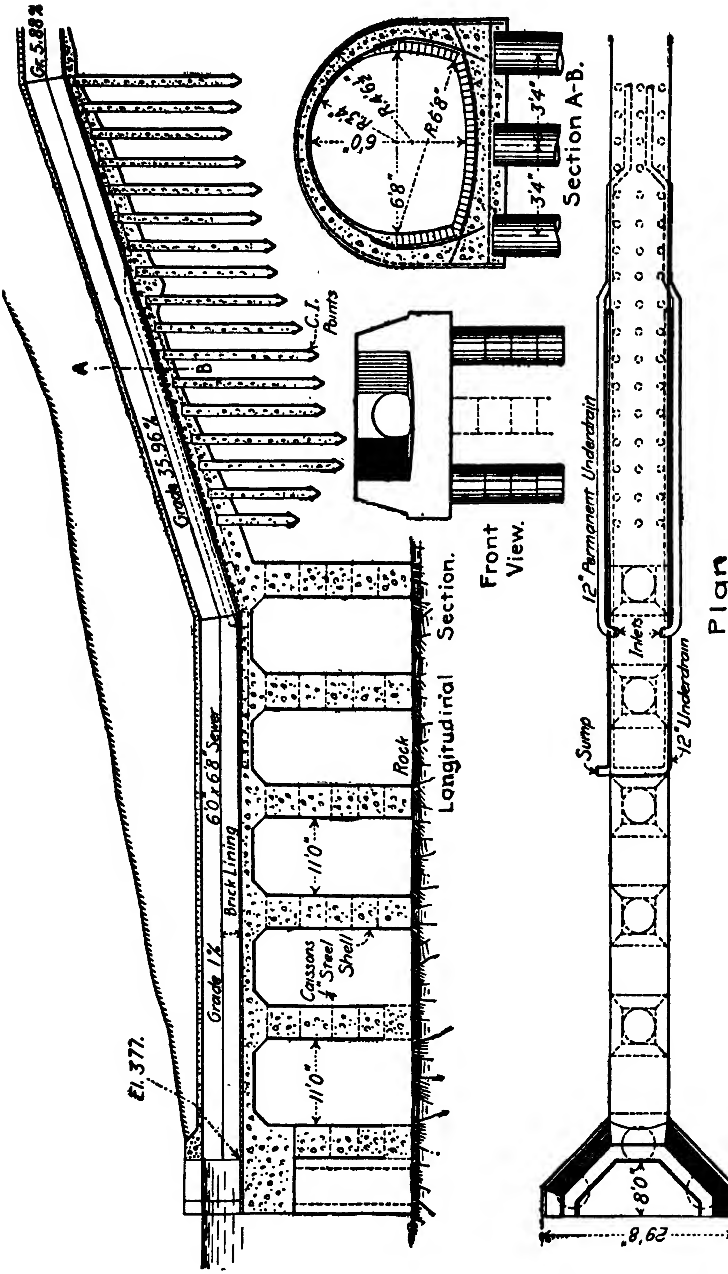


FIG. 260.—Northwestern outlet structure, Louisville, Ky.

between piers. The minimum thickness of the concrete of the invert of the sewer, between the column caps, is 2 ft. The illustration shows the supports as planned; as a matter of fact, these columns proved so much superior, in this work, to the reinforced-concrete piles, that the last four bents of the latter were omitted and a column substituted for them.

The drop chamber is carried on Simplex concrete piles with four ¾-in. twisted reinforcement bars extending from their top into the concrete of the invert of the chamber.

The outlet of the Broadway outfall sewer (Fig. 261) in the Borough of the Bronx, New York City, is 57 ft. long on its outer side along the

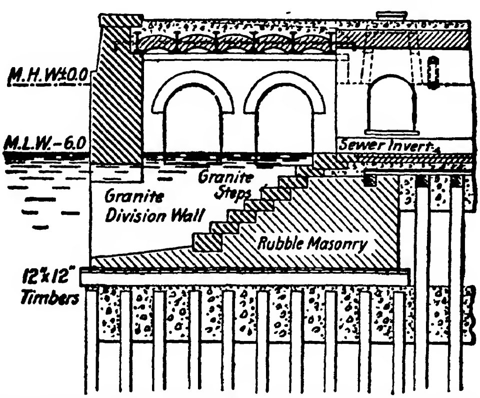


Fig. 261.—Outlet of Broadway outfall sewer, Borough of the Bronx, New York.

river, 41 ft. long on the inner side where the sewer joins it, and 21½ ft. wide along the cross-section shown in the illustration.[1] It has a total height of 22 ft. and is constructed of granite ashlar masonry with a heavy concrete roof. The invert of the end of the outfall sewer is at mean low-tide level. This sewer has a twin semicircular section. The sewage escapes through four openings in the front wall, each 8 ft. wide and 8 ft. high, and as the arch of the opening in each case is below mean low water, it is expected that no floating matter will leave the outlet chamber.

The outlet of the 92nd Street sewer in the Borough of Brooklyn, New York City, shown in Fig. 262, includes an increaser chamber 80 ft. long extending from the end of an 11-ft. sewer where it emerges from a tunnel to a triple sewer having three basket-handle sections carried out on a riprap embankment far enough for the sewage to be discharged into a portion of the Narrows having swift tidal current. The whole structure is very heavy, owing to the strong current to which it is

[1] *Eng. Record*, 1905; **52**, 550.

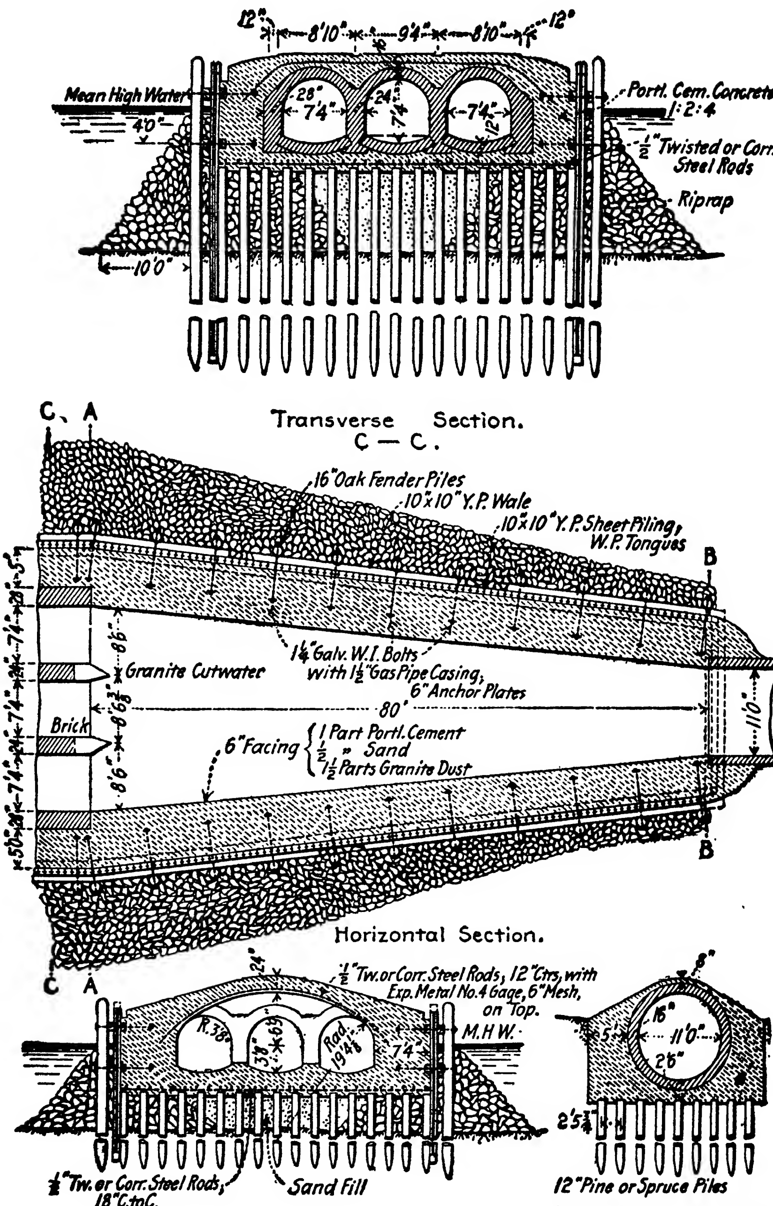


Fig. 262.—Increaser chamber, Brooklyn, N. Y.

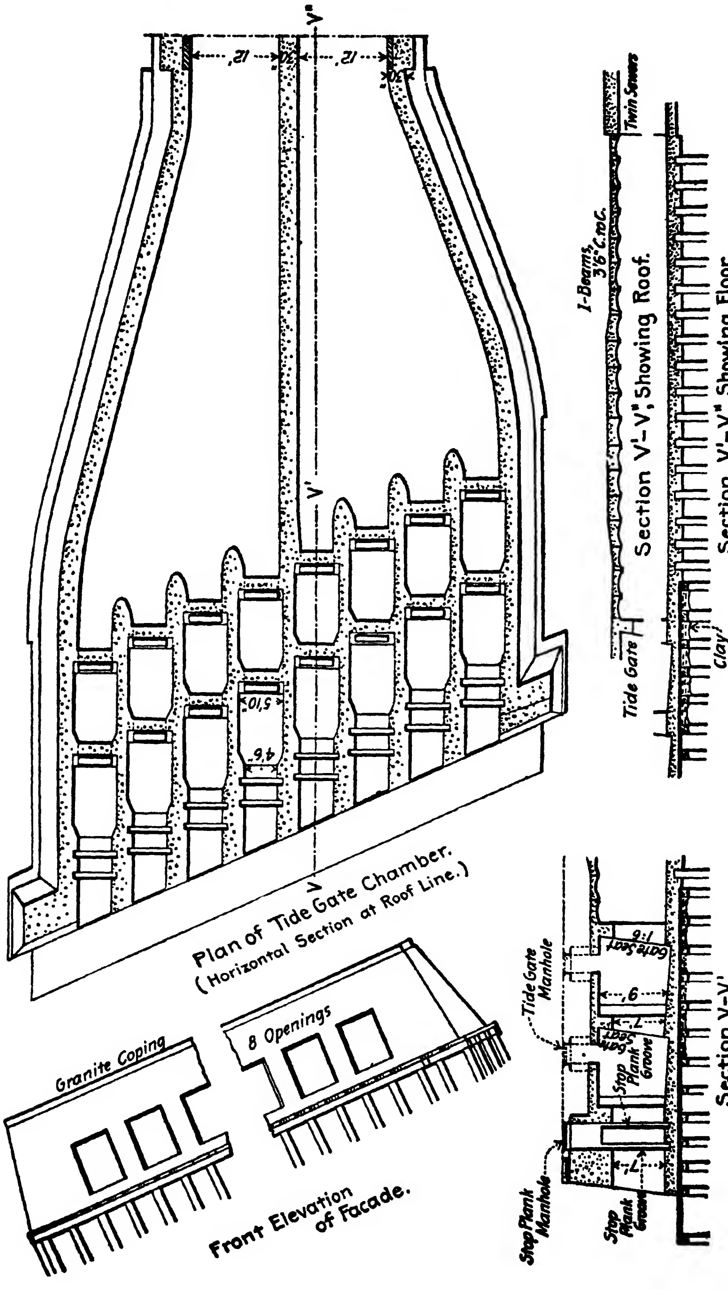


FIG. 263.—Storm-water outlet at Washington, D. C.

subjected, and also to the fact that it may be utilized in the future by the Municipal Department of Docks and Ferries. The bottom at the site of the outlet is coarse shingly gravel, with a lower stratum of compact sand and gravel.

Figure 263 is the outlet structure for storm-water of the high-level intercepting sewer in Washington. The water is brought to the structure in two 12-ft. channels with arched masonry roof. The outlet structure is provided with a roof of concrete between I-beams spaced 3

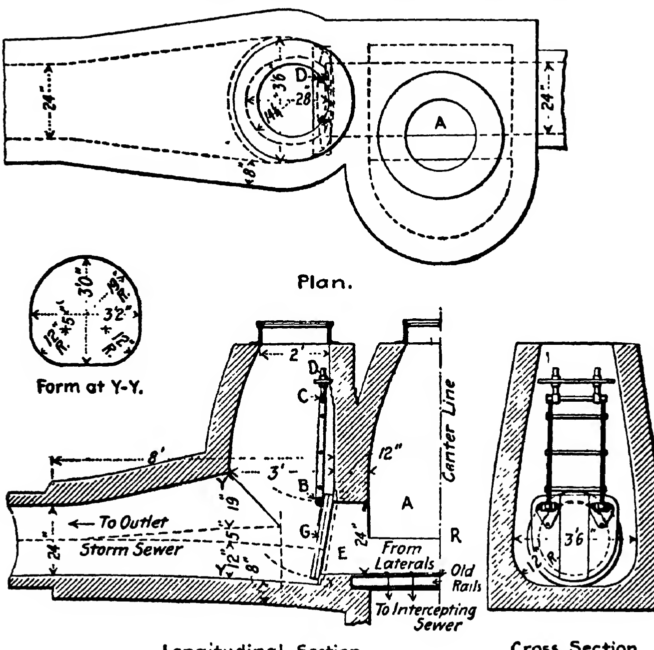


FIG. 264.—Tide gates at Providence, R. I.

ft. 6 in. apart. The outlet has a longitudinal wall 30 in. wide which supports the inner end of these beams along the center line of the structure. The general arrangement of the structure is shown so well in the illustration, from drawings furnished by Asa E. Phillips, Superintendent of Sewers of the District of Columbia, that no explanation is necessary. The entire structure is carried on piling spaced 3 ft. 6 in. on centers in each direction for the most part.

TIDE GATES

Wherever an outlet ends at a body of water subject to considerable fluctuations in level and it is necessary to prevent this water from enter-

ing the sewer, a backwater or tide gate is employed. This consists of a flap hung against a seat which inclines backward as it rises. The hinges may be at the top in case the gate consists of a single leaf, as is usually the case, or they may be at the side, in case the gate consists of two leaves.

One of the earliest types of large tide gates to work satisfactorily was that designed by Otis F. Clapp while in charge of the sewer department of Providence, R. I., of which place he subsequently became city engineer. This is shown in Fig. 264.[1] Ordinarily the entire flow from the 24-in. lateral sewer dropped through the rack *R* in the bottom of the chamber *A* into the intercepting sewer at a lower level. When the volume of sewage became too great for the intercepting sewer, it rose in the chamber *A* and swung open the gate *G* so as to obtain an outlet through the storm sewer into Narragansett Bay. The gate *G* revolved about its axis, *B*, and also about the axis, *C*, so that it moved freely even with a slight flow of sewage from the chamber *A*. When the tide backed up to the storm sewer, the gate was pressed firmly against its seat. The adjustment of the gate in position was readily made by means of the nuts *D*. The unusual feature of this design is the use of very long links for holding the gate.

For a number of years, the larger tide gates in Boston were frequently hinged at the sides. Each gate consisted of two leaves, and as the seat was inclined with the top inward, as usual, when the gate opened it rose slightly as well as moved outward. Consequently it tended to fall back again when the pressure of the outflowing storm water and sewage decreased. In order to make certain that this closing should take place, it was customary to hitch to the back of each leaf a "bridle chain" hanging loosely from a substantial eye in the roof of the gate-chamber. On the lowest part of the chain as it hung between the roof and the closed gate was a heavy weight. This bridle chain tended to close the gates when they were open. This type of gate was known as the "barn door" and has now been abandoned on account of the number of adjustments which were necessary to keep it in condition and because the bridle chains accumulated large masses of floating substances which interfered with their proper operation.

The type of tide gate and chamber now used in Boston is shown in Figs. 265 and 266. The rubber gasket of the wooden gate rests directly on the end of the cast-iron seat. The type of gate illustrated in Fig. 265 is regularly made for 12-, 18-, 24-, 36-, 48-, and 60-in. outfalls. The timbers are held together by vertical binding rods, and in the lower part of the flap a number of displacement weights are inserted, which make the gates very sensitive to back pressure. A larger gate of the same general type, designed for use in Boston, is shown in Fig. 266.

[1] *Eng. Record*, 1896; **31**, 241.

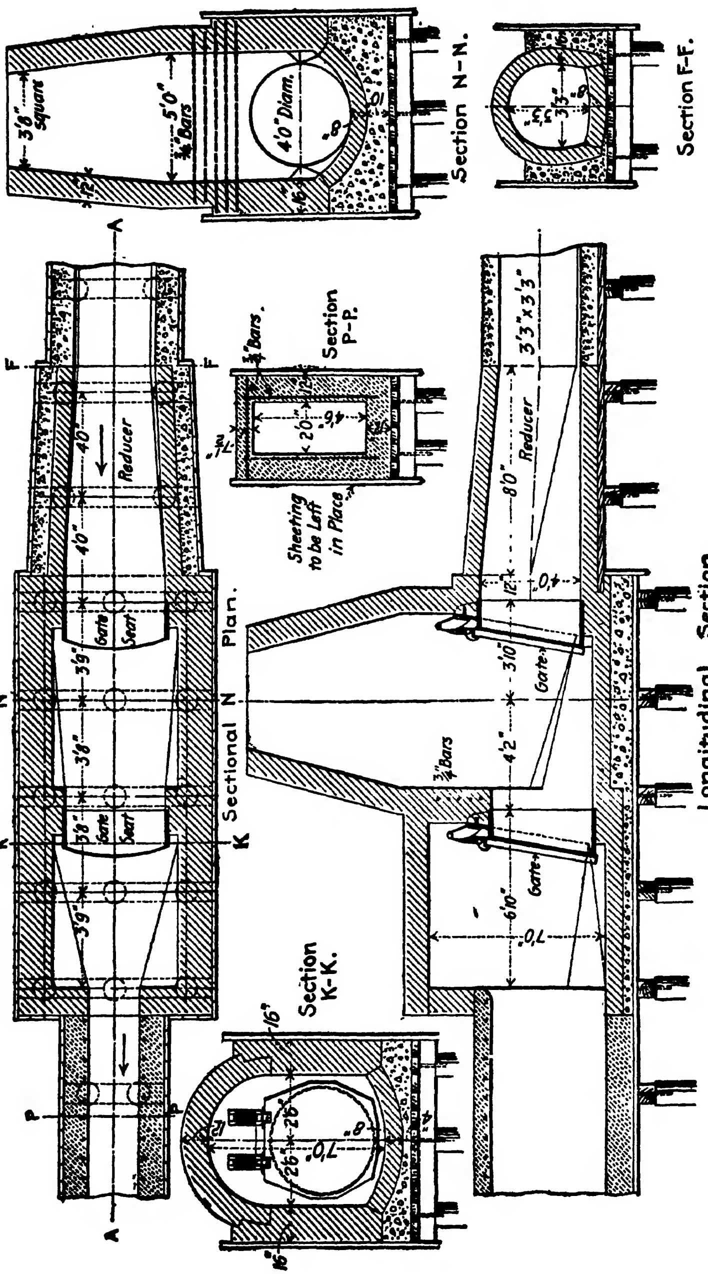


FIG. 265.—Boston tide gates, 1911 type.

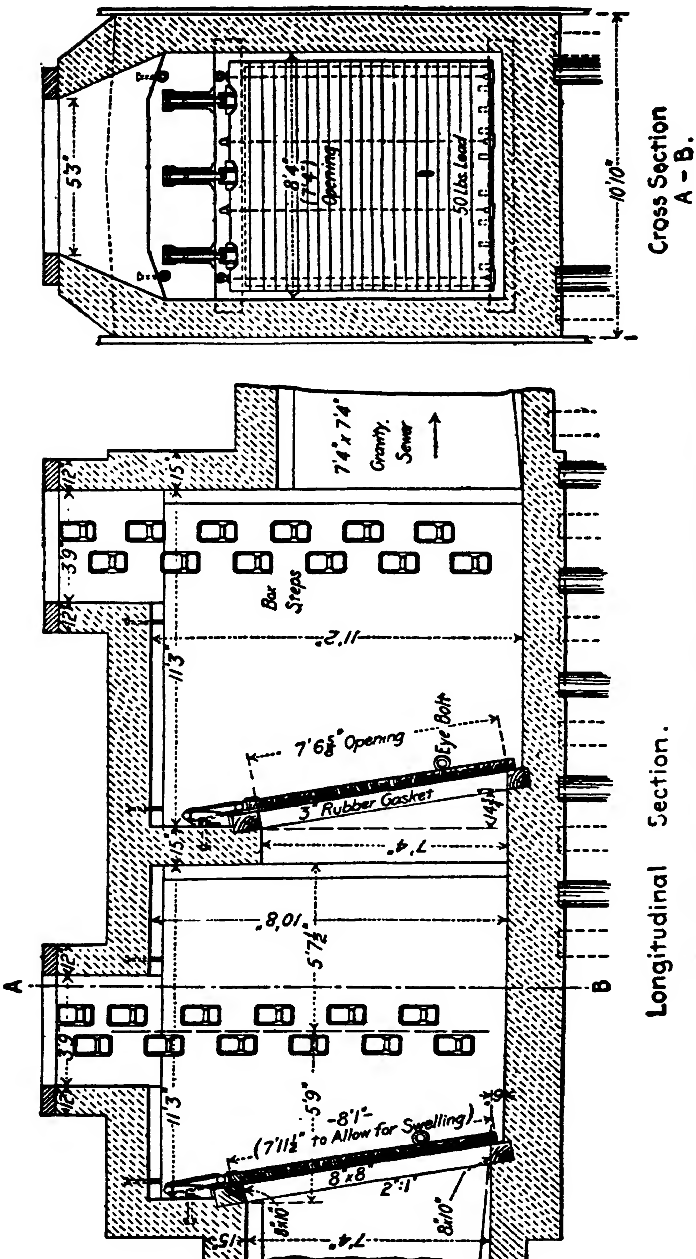


FIG. 266.—Tide-gate chamber, Boston, with McNulty gates (patented.)

As timber seats are used, these gates may be made of any desired size. The hinges are fitted with links to prevent breakage of either the hinge or gate should floating matter be caught between the frame and the gate. The Boston gates are strong though light, flexible, and well balanced.

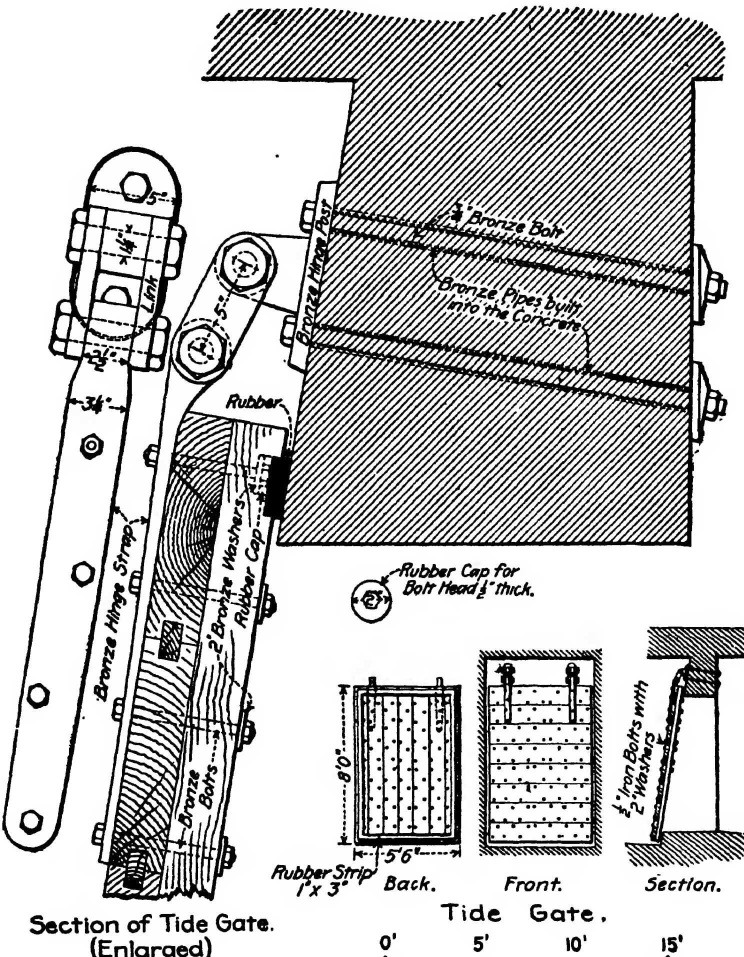


FIG. 267.—Details of Washington tide gates.

They are manufactured by the McNulty Engineering Company of Boston.

The tide gate used in Washington in the structure illustrated in Fig. 263, is shown in Fig. 267. Asa E. Phillips informed the authors

that these gates are made of double cross-lapped 3- by 12-in. Georgia pine, shipped directly from the southern mill where it is cut and kept under a damp cover until ready to place. The contact is made on the concrete gate seat by a rubber strip 3 in. wide and 1 in. thick, set half into the wood. These gates have been very effective, requiring scarcely any attention, and have always been substantially watertight. They required no renewal or repair for 5 years after their erection, and very little for a number of years longer.

The operation of tide gates by hand has been attempted at times, as at Hoboken, N. J., where there were three thus served in 1912 and one which was a simple automatic flap gate like those in Boston. James H. Fuertes found in that year that there was an attendant at each manually operated gate all the time, 12-hour shifts being in force, and each man followed a system of his own in managing the gates. At one place the gates are opened 1 hour after high tide and closed 1 hour after low tide, with some variation during very high or low tides. At another place the gates are opened from 3½ to 4 hours after high tide and closed from 2 to 2½ hours after low tide. Observation showed Fuertes that the proper time to open the gates was directly after high tide and for closing them 2 hours after low tide, and that automatic gates would probably give better service than manual operation of the kind likely to be provided.

VENTILATION

For many years the provision of special structures to aid the ventilation of sewers was one of the most troublesome tasks of the designer. The gravity of the problem is probably not appreciated today, when the necessity of good grades and construction is so generally recognized that the conditions which frequently faced a city engineer not many years ago are hardly to be believed. When the sewerage systems frequently contained old sewers which had either been so constructed as to cause the formation of banks of sludge and pools of septic sewage, or had been allowed to fall into such a dilapidated condition that the same evil results followed, it is not surprising that engineers as well as the general public had good reason for believing that there was such a thing as "sewer gas." There were a number of books written on the subject of this "gas" and it was naturally seized upon as an explanation of various diseases of city dwellers, although the relation between the two was never shown. One result of the offensiveness of the air in many sewers was the practically universal use at one time of main or house traps between the sewers and the plumbing systems in buildings. The presence of these traps made it impossible to ventilate the sewers through the soil pipes within the buildings.

In some cases, however, ventilation was afforded by a pipe run up from the house drain, just outside the main trap, and generally carried above the roof on the outside of the building, although this position was often impracticable and substitutes were adopted, one of the worst being to have the ventilating pipe terminate in the "area" in front of the house, a foot or two above the ground. Many other methods of ventilating the sewers were also tried. One of the most obvious, which is still extensively employed, was to use perforated covers for the manholes. At one time perforated trays of charcoal were placed in the shafts of the manholes, in the belief that the sewer air in passing through them would be disinfected. In order to increase the draft up the vent pipes on the faces of the buildings, many kinds of cowls to surmount them were invented. Sometimes the riser was provided with a bent pipe admitting fresh air to the interior in a vertical direction, with a gas jet in the center of the vertical portion of this inlet, so that the flame of the jet drew a current of air constantly into the riser and also helped the upward draft in it from the sewer. Ventilating street lamps have been installed, particularly in British cities, in which the air was drawn from the sewer in a pipe and sucked up a shaft resembling an ordinary gas lamp post, by the draft of a gas lamp, through the flames of which the sewer air had to pass before it could escape.

With the steady improvement in the construction of sewerage systems and the abandonment or rebuilding of the old sewers which were defective, the annoyances due to foul odors became so rare that it occurred to many engineers that the necessity for main traps no longer existed where the sewers were in good condition, and that the ventilation of sewers would be greatly helped by the omission of such traps. This opinion led to a number of investigations of the real nature of sewer air. One of the first of these was made by J. Parry Laws at the direction of the London County Council. He found that the bacteria in the sewer air were related to those in the external air and not to the bacteria of sewage. The inference he drew from this was that no matter how many germs of disease might be in the sewage they were not likely to enter the air above it unless the sewage splashed violently, as would be the case at the entrance of a branch sewer into a trunk sewer at a considerably different elevation, or where the sewage fell down a manhole shaft. There was little probability, in his opinion, of bacteria passing from the walls of a sewer to the air, after the sewage level had fallen, because he found in one experiment that an empty pipe sewer, to which large numbers of bacteria must have been attached, effected no increase in the bacteria in a current of air sent through it. Although his experimental evidence was contrary to the probability of sewer air containing disease germs not found in external air, he nevertheless drew the following conclusions:

Although one is led almost irresistibly to the conclusion that the organisms found in sewer air probably do not constitute any source of danger, it is impossible to ignore the evidence, though it be only circumstantial, that sewer air in some cases has had some causal relation to zymotic disease. It is quite conceivable, though at present no evidence is forthcoming, that the danger of sewer air causing disease is an indirect one; it may contain some highly poisonous chemical substance, possibly of an alkaloidal nature, which, though present in but minute quantities, may nevertheless produce, in conjunction with the large excess of carbonic acid, a profound effect upon the general vitality.

In 1907 Dr. W. H. Horrocks found at Gibraltar that where sewage fell vertically the air in the sewers contained the colon bacillus and various streptococci. He also found that it was possible to put easily recognized forms, such as B. prodigiosus, into sewage and recover them from the air of the sewers, into which it was assumed that they entered by the bursting of bubbles of gas rising from the sewage, from splashing of falling sewage, or from the drying of the sewage left on the walls of sewers when the depth of flow dropped. Other experiments of the same nature were made about the same time by Dr. F. W. Andrewes, and were recorded in the report of the Medical Officer of the Local Government Board for 1906–1907.

Prof. C.-E. A. Winslow found in 1908, in an investigation for the Master Plumbers Association of Boston, that while the results of the investigations of Horrocks and Andrewes were undoubtedly correct qualitatively, the number of bacteria thrown off from sewage was so extremely small that the local infection of the sewer air was of no importance whatever. The general air of the house drains was found to be singularly free from bacterial life. Even near the points where splashing occurred there were only four times when intestinal bacteria were found, which led Winslow to conclude that, so far as infection is concerned, sewer air is not to be held responsible for the spread of infectious diseases.

In a later paper, "The Atmosphere and Human Health,"[1] Winslow states that "as far as the transmission of disease microbes through the general medium of the atmosphere is concerned, epidemiological evidence is overwhelmingly negative." If the effect of splashing of sewage may be considered as somewhat similar to mouth spray discharged in speaking, sneezing, or coughing, it seems probable that its effect could not extend to the air outside of the sewer itself. Winslow says that this

. . . mouth spray is a fine rain which settles quickly to the ground, not a vapor which disseminates itself through the air, and Robinson and the speaker have found that the vast majority of the bacteria discharged in

[1] *Trans. Am. Soc. C. E.*, 1925; **89**, 316.

coughing, sneezing, and loud speaking disappear from the air beyond the radius of a few feet.

He further reports that considerable attention has been paid during recent years to the relation of disagreeable odors to health. The belief that organic emanations in noxious odors may be toxic or predispose to disease has been definitely proven to be without foundation in fact. The matter has been thoroughly studied from the standpoint of poorly ventilated rooms, rather than as directly applicable to sewer gases, but the findings throw light upon the causes of the feelings of discomfort accompanying the breathing of foul air under conditions of inadequate ventilation. Here the decrease in oxygen and the increase in carbon dioxide and in products of organic decomposition (producing "body odors") have been shown to be insufficient cause for the ill effects experienced, but these are due to an increase in temperature and humidity. Tests made by the New York State Commission on Ventilation indicate a greater disinclination to physical exertion accompanied by falling off of appetite under conditions of poor ventilation. At Yale, experiments indicate a retardation of growth of young guinea pigs when exposed to odors from the decomposition of organic matter.

It is the general opinion of engineers today that when a sewerage system is well designed, carefully built, and properly maintained, the sewage passes from the houses to the disposal works or outlets in a steady course which affords little opportunity for the subsidence of suspended matter or the occurrence of offensive putrefaction and fermentation. Unfortunately, accidents occur which may cause sewage to collect in pools or at least to lose velocity to such an extent that more or less of the solids will settle to the invert. When this happens the sewer is likely to become offensive. It follows from this that the maintenance of a sewerage system should always be well provided for, and those in charge of the work should appreciate the importance of investigating every complaint which is made regarding foul air from the system. These disturbances of the proper operation of the sewer network are generally considered as the only excuse for retaining the main traps on house connections, which are now believed by most engineers to be the main obstacle to the efficient ventilation of sewers. In other words, the recent improvements of sewerage systems, effected by a small expense for better engineering and more rigid supervision of construction, have saved a considerable expense in ventilating appliances and a great deal of annoyance to property owners on account of disagreeable odors. Winslow stated in 1909, in a letter read before the Boston Society of Civil Engineers:

While we are right in spending money for plumbing which is free from gross defects, we are not as obviously justified in recommending large

expenditures for refinements like back ventilation and intercepting traps between the house and the sewer. The trapping of ordinary fixtures does away with most of the possible dangers of sewer gas. There are plenty of traps which will give a reasonable degree of security against siphonage without back ventilation.

One of the best proofs that these conclusions are correct is the fact that the laborers engaged continually underground in the sewers of Paris, who are kept under strict observation, show no indication whatever that their work in sewer air has any effect on their health.

There are a few authentic cases of loss of life in this country due to sewer air. One of these is mentioned on page 569, and occurred near the outlet of the Los Angeles outfall sewer. Another happened in a gate chamber of the intercepting sewerage system in Syracuse and a third happened in 1906 in a dead end in San Francisco. In each case it is probable that the gases given off by the changes in the composition of the sewage, which are usually carried along within the sewage to a certain extent, were liberated and collected at the places where the accidents occurred. The actual composition of the gases varies with the character of the sewage but may include air, illuminating gas, vapor from gasoline, carbon monoxide, carbon dioxide, and methane in varying amounts. Such accidents as have occurred have been due to the presence of some of these gases and indicate that the same precautions should be observed in entering manholes as are observed in entering wells, *viz.*, the lowering of a miner's light to be certain that the air is safe to breathe, and avoiding the use of a flame unless protected as in the miner's lamp. The difficulties encountered from foul odors will be more in evidence at sewage treatment plants. The methods employed for controlling these by utilizing the gases formed are discussed in Vol. III.

The movement of air in sewers is due to a number of causes, such as the difference in unit weight between the outer air and that in the sewers, the difference in elevation of the various openings between the sewers and the external air, the flowing of the water through the sewers which tends to move the air resting on the liquid, and the effect of the wind on the openings into the sewers, particularly the outlets of large sewers.

Theoretically, the most effective openings for ventilation should be those in the manhole covers. The connections through which sewage and rainwater are delivered to the sewers from houses are likely to be filled from time to time by the discharges from those properties, while the traps used on many street inlets and catchbasins, if they are in proper condition, will be sealed by the water within them so that no air can enter or escape there.

The effect of the temperature inside and outside the sewers upon the ventilation of the latter usually depends upon moderate differences in

temperature and the unit weights of air due to these temperatures, although the difference may be large in winter. The movement of the sewer air is theoretically toward the end of the laterals, since they are at higher elevations than the trunk sewers and the warm sewer air which is endeavoring to escape during a considerable portion of the year will naturally rise, while the colder outer air will enter at the lower openings of the sewers. Practically, however, it seems to be a fact that the wind and the drag on the sewer air due to sewage flowing down grade have some effect at times in checking the upward motion of the air.

While these views are theoretically sound, they were shown to be of little practical importance by a very thorough investigation made in Leicester, England, in 1898–1899, by the borough engineer, E. G. Mawbey. When he took charge of the sewerage system it was provided with ventilated manholes and lampholes at the rate of about 37 to the mile. Many complaints of foul odors were made, which finally led to the adoption of a policy of closing the manhole covers on ascertaining that odors actually came from them, and running ventilating pipes up the adjoining building if permission to do this could be obtained. This was in accordance with the practice of many other English cities. In order to ascertain how much circulation was really obtained through these manhole covers, Mawbey carried out many experiments. In a typical instance a 6- by 4-in. shaft was erected between two manholes 150 ft. apart. Anemometer tests showed that in both manholes the outward flow of air, after the shaft was erected, exceeded the inward flow in the proportion of 69 to 20 in one case, and 41 to 19 in another. In another case where a complaint was made of odors at a manhole at the intersection of two large sewers, two stacks of 9-in. stoneware pipe were erected side by side and the manhole cover left open. Anemometer tests showed that the upward flow of air with the double shaft amounted to 505,000 cu. ft. per day, and although it was only about 94 ft. distant from the manhole cover, there was an upward flow through the latter of 40,500 cu. ft. a day, while the inward flow was only 18,000 cu. ft. per day. The cover was still a nuisance and was made tight. Many similar experiments were made, which showed that the column of air in the manholes was too low to make the ventilation through their covers a matter of any importance. This confirms the general American opinion that it is best to ventilate the sewers through the house connections, when the sewerage system is in good condition and there are good plumbing regulations which are enforced strictly.

The authors have found that many complaints of offensive odors from sewers have been due to the discharge into them of industrial wastes, such as refuse from gas works. In one case the trouble was traced to crude oil, which had escaped from the underground piping of a forging plant and percolated into the sewer through leaky joints. Packing-

house refuse is particularly offensive, and if discharged into a sewer having a sluggish current it may be the cause of foul odors. Many times foul-smelling gases are forced through perforated covers of manholes by steam discharged into the sewers. In fact, odors are more likely to be given off from hot than from cold sewage.

CHAPTER XVIII

PUMPS AND PUMPING STATIONS

In the design of sewerage works, it may be necessary to provide for pumping where the sewage or storm water is collected at so low an elevation that discharge by gravity is impossible, as at Washington; to reach a desirable purification site, as at Baltimore; to lift the sewage from areas too low to drain into the main system by gravity; or to force water into streams or tidal inlets receiving sewage, which would become offensive unless flushed in this way.

Whether the sewage shall be lifted at one or more points is usually a matter to be settled by comparing the fixed and operating expenses corresponding to different plans. The operating expense of raising all of the sewage at one point is less than that of pumping at two or more points. On the other hand, if all of the sewers are made to drain by gravity to one place, their cost may be greatly increased on account of the deep cuts and large cross-sections necessary in order to obtain satisfactory velocities of flow. Various projects must often be considered, both with and without pumping, and the extra cost necessary to drain to one point, together with the cost and the capitalized annual charges for operation and depreciation of the pumps must be compared with similar charges for a project with two or more stations.

PUMPS FOR SEWAGE

The requirements for pumps to be used in elevating sewage or drainage water differ considerably from those for pumping clean water. The lift is comparatively small; simplicity and reliability are of more importance than high efficiency; valves (if required) and passages must be able to pass solid matter with a minimum of clogging or repairs; and the possibility of automatically starting and stopping pumps to take care of variations in flow is often a controlling item.

Types of Pumps.—There are four types of pumps which may be used for sewerage and drainage work, namely, reciprocating pumps, centrifugal pumps, screw pumps, and pneumatic ejectors.

RECIPROCATING PUMPS

As the name indicates, a reciprocating pump is one in which water is displaced by the forward and backward motion of a piston or plunger

within a cylinder. It is a *positive-displacement* pump, and each stroke of the plunger or piston necessarily displaces a definite volume of water. The actual quantity pumped per stroke is less than the displacement by the amount of the *slip*, or quantity leaking through the valves and past the plunger or piston.

Reciprocating pumps are of two general classes, commonly known as *power pumps* and *steam pumps*. A *power pump* is driven from an external source of power, such as an electric motor, an internal-combustion engine, or a steam turbine, often through belting or gears; while in a *steam pump* the power is transmitted from the steam pistons directly to the pump. In the larger sizes of steam pumps, the employment of cranks and flywheels is common: such pumps are commonly called *pumping engines*.

The use of reciprocating pumps for sewage or drainage water usually necessitates:

a. Screening the sewage for the removal of solids large enough to clog the pumps.

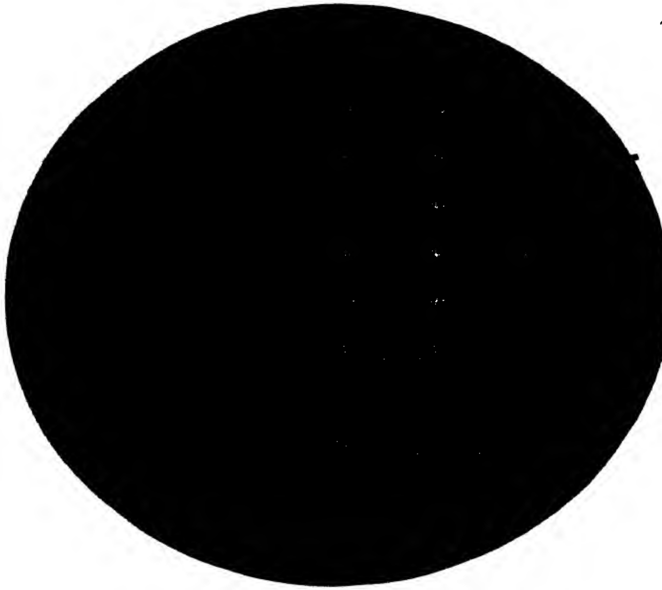
b. Settling the sewage to remove grit which would cause rapid wear.

c. A design of pump with large passages and with valves capable of passing the smaller solids without clogging; and providing for repacking plungers and for taking up wear without too great difficulty.

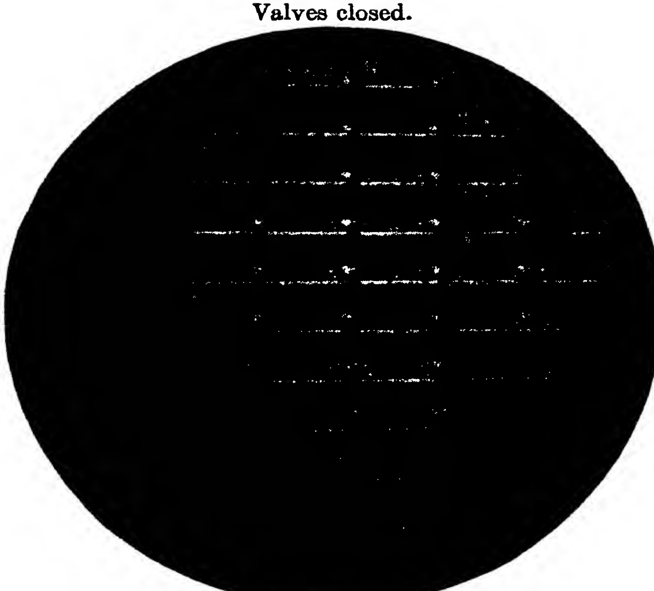
A considerable variation in the speed of a steam pump is possible, with a corresponding variation of capacity, and changes in the sewage flow can be cared for to a certain extent by regulating the speed; but a power pump must generally be operated at a constant speed, and changes in the flow must be met by changing the number of pumps in operation, unless it is possible to equalize the rate of flow by interposing a large suction well or reservoir.

Pump valves for sewage must be either of the ball or of the flap type. Ball valves are rarely used except for sludge. Flap valves, which are generally used in sewage pumps, are either actually hinged or attached to the valve deck so as to move as if they were hinged. Flap valves frequently cause much trouble because sticks and rags are caught on their seats and hold them open. Hinged flaps are more often used than the simple flap pattern; they have a leather or rubber disk held between metal plates, the top plate having an arm running sideways to a hinge connection with the valve seat. The valve deck and valves of the Baltimore sewage pumps, built by the Bethlehem Steel Company, and described briefly later in this chapter, are illustrated in Fig. 268.

Reciprocating pumps for sewage are rarely used. In earlier years they were employed in a number of large plants because of the high duty which could be obtained by triple-expansion or compound pumping engines; but recent improvements in centrifugal pumping machinery



Valves closed.



Valves open.

Fig. 268.—Valve deck, Baltimore sewage pumps.

have made it very nearly, if not quite, as economical in operation as the best steam pumping engines. The latter are much more costly, occupy far greater space, and require more attendance for their operation, than do centrifugal pumps.

There are several large pumping stations for sewage in which high-duty reciprocating pumps were installed years ago, which are still operating satisfactorily and economically. Those at Baltimore and Boston may be noted especially.

CENTRIFUGAL PUMPS

Description.—A centrifugal pump consists of two principal parts, a fixed *casing*, and a rotating *impeller* or runner, provided with one or more

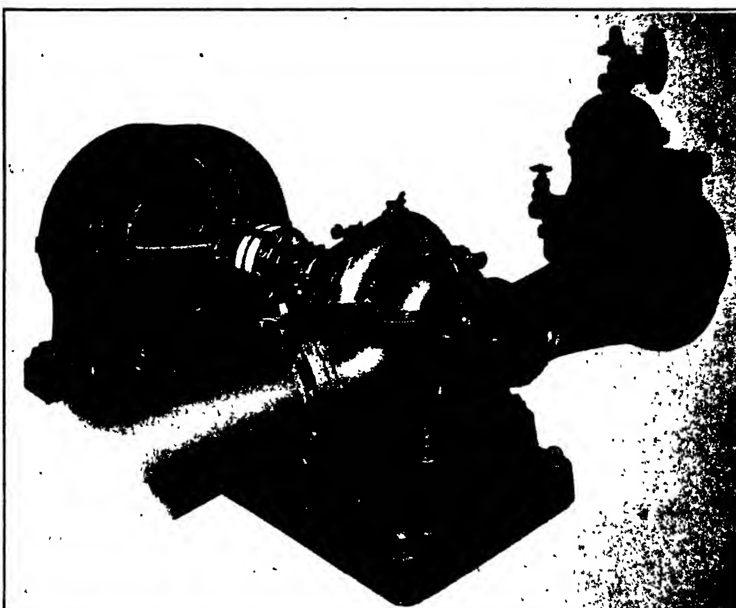


FIG. 269.—Horizontal centrifugal pump, double suction, single stage.

vanes. Water is admitted to the impeller near the center or eye, either on one side (single suction), or on both sides (double suction). The vanes may be held between side plates which form part of the impeller (closed impeller), or the plates may be omitted or fastened rigidly to the shell of the pump, so that the impeller consists substantially of the shaft and vanes only (open impeller). When the impeller is rotated the water is thrown off at the periphery with a considerable velocity; and as the

water is confined within the casing (*volute*), the velocity is largely transformed into pressure.

It is impracticable to build up sufficiently high velocities with a single impeller to overcome very high heads; but any lift can be pumped against by arranging a number of pumps in series, each taking its suction from the discharge of the preceding pump. When clean water is to be handled, two or more impellers can be mounted on a single shaft, and if the casing is properly shaped to guide the water from one impeller to the next, a *multistage* pump is obtained. Such a pump is not suited to handling sewage, but it is seldom that heads are high enough in sewerage

FIG. 270.—Vertical centrifugal pump, single suction, single stage. (*Morris Machine Works.*)

work to require more than a single stage. Where comparatively high heads are involved, two or more separate pumps can be arranged in series.

The early centrifugal pumps were crude and inefficient. Even after considerable efficiency had been obtained in pumps for water, furnished with closed impellers and guide vanes, it was still necessary to use open impellers for sewage in order to reduce the clogging to a minimum, and the efficiency of such pumps was low. In recent years, however, great improvements have been made, especially in the design of impellers of the "nonclogging" or "trash pump" type, and it is now possible to obtain very good efficiencies in pumps which are remarkably free from clogging, even when handling unscreened sewage.

The size of a centrifugal pump is defined as the diameter of the discharge nozzle.

Centrifugal pumps are called vertical or horizontal, depending upon the position of the shaft.

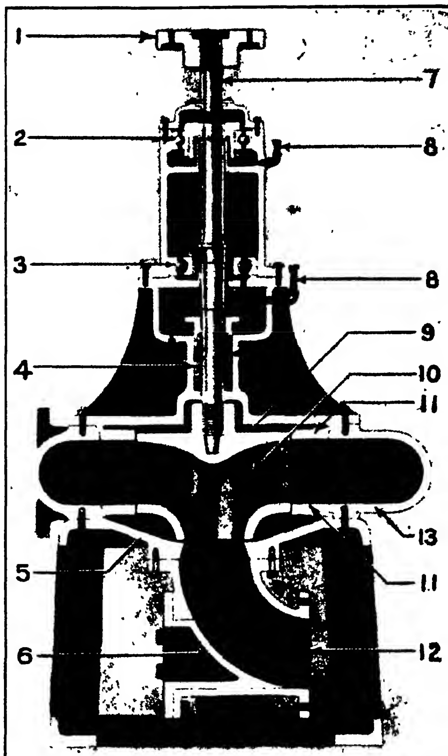


FIG. 271.—Section of vertical centrifugal pump for sewage. (*Fairbanks-Morse.*) This section of a sewage pump shows how the impeller (10) and the flexible coupling (1) are mounted on the shaft (7) which is carried by a combined radial and thrust bearing (3) and guided by a radial bearing (2). Where it passes through the stuffing box, the shaft is protected by a bronze sleeve (4). The bearings run in oil the level of which is indicated by the gages (8). The suction elbow (12) with a clean-out hole (6) is attached to the lower or suction head (5) which is cast as part of the base. The volute casing (13) which may be equipped with adapter rings (11) for impellers of various diameters supports the upper head (9) and bearing housing.

Figure 269 illustrates a typical horizontal double-suction centrifugal pump, and Fig. 270 a vertical pump with single suction. A section of a pump of the latter type is shown in Fig. 271. Figures 272 to 277 illustrate forms of so-called "non clogging" impellers especially designed for handling sewage and drainage water.

These and several other forms of "nonclogging" impellers have been developed especially to make it possible for the pump to pass solid objects of considerable size,[1] and also to minimize if not eliminate the possibility

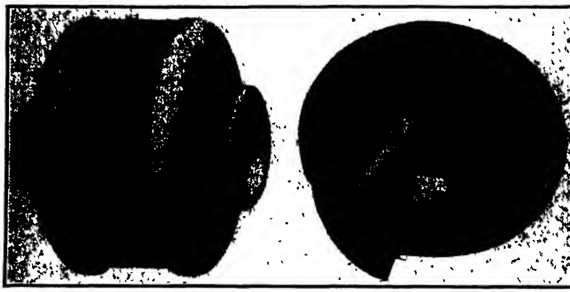


FIG. 272.—General view and section of closed impeller used in Fairbanks-Morse sewage and trash pumps.

of clogging by rags or other flexible substances which might catch upon the vanes of the impeller. They are particularly advantageous when pumps must be operated with a minimum of attendance, and when

FIG. 273.—Impeller of Morris Machine Works non clogging pump.

screening is either impracticable or cannot be depended upon to remove the clogging substances. Obviously, screens should not be omitted when a 4-in. or smaller pump is used.

[1] For pumps not exceeding 12 in. in size, the common requirement has been that they should pass spheres one size smaller than the pump.

The requirements of and the experiences at New Orleans have had much to do with the development of pumps of this type. Here all the sewage has to be pumped to the Mississippi River, and as the height of water in the river varies, it is necessary to have pumps which can operate satisfactorily against variable heads. The earliest pumps, about 1904,

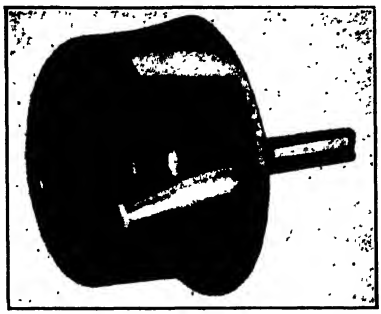


FIG. 274.—Impeller of American Well Works nonclogging pump.

had impellers containing three outlet passages, the spaces between these being either vacant or blanked off. These pumps were capable of passing large solids, but gave considerable trouble from vibration, the efficiencies were low, and fibrous material would accumulate upon the

FIG. 275.—Impeller of Yeomans nonclogging sewage pump.

vanes. Gradual development since that time has resulted in the various types of "nonclogging" impellers illustrated.

Many centrifugal pumps for sewage with impellers of the ordinary type are in service, and others are being and will be installed. Some pump makers who furnish nonclogging impellers believe that these are of little if any advantage in the smaller sizes of pumps, since with them the

sewage must be screened in any event. Others claim that the passages in the larger pumps are adequate for such solids as may pass coarse racks, and that nonclogging impellers are advantageous only in the smaller sizes.

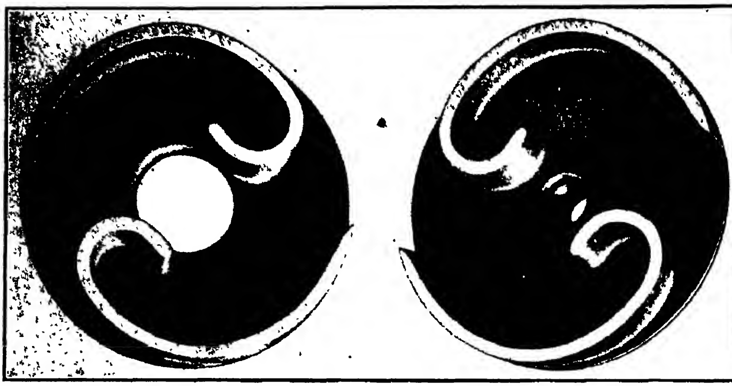


FIG. 276.—Impeller of "Non-clog" sewage pump. (*Chicago Pump Company.*)

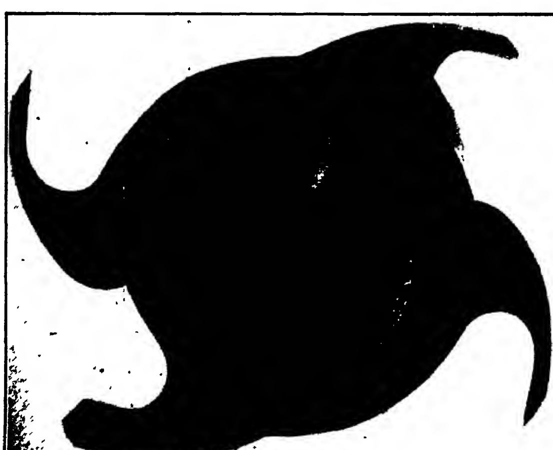


FIG. 277.—Impeller of Worthington trash pump.

Power for Centrifugal Pumps.—Any source of power can be utilized for driving centrifugal pumps, by direct connection of the shafts if the prime mover runs at the proper speed, or by the use of belts or gears if the prime mover runs at a different speed from that of the pump.

Vertical centrifugal pumps are most conveniently operated by electric motors. The employment of any other type of power would generally involve the use of bevel gears or of belting containing a quarter-turn, neither of which is desirable. In the case of a large pump, however, it is practicable to provide direct connections from the cylinders of a steam engine to cranks rotating in a horizontal plane. This has been done in some cases, notably in the large pumping stations of the (Boston) North Metropolitan Sewerage Works, where the three cylinders of triple expansion engines have been set 60 deg. apart to drive the vertical shafts of large centrifugal pumps, having impellers 7 ft. 6 in. and 8 ft. 3 in. in diameter.

Horizontal pumps may be conveniently driven from any source of power. The most common methods are:

1. By electric motors through direct connections
2. By steam turbines through reducing gears
3. By Diesel or other internal-combustion engines, either through direct connection (for slow-speed pumps) or through belting
4. By electric motors through belting (usually for small pumps)

Electric power is particularly advantageous for this service from every point of view, except that of cost and, possibly, of dependability; and in some cases the cost may be as low as, or even less than, that for any other kind of power.

Priming Centrifugal Pumps.—A centrifugal pump, unless set below the level of the water to be pumped, cannot prime itself, and must be filled either by exhausting the air from the casing, or by admitting water from some external source until the air has been expelled. Once in operation, a centrifugal pump can continue to function with a moderate suction lift, preferably not exceeding 15 ft.; but if air collects in the pump casing, or even in comparatively small pockets in the suction piping, the discharge of the pump may be materially reduced or may cease altogether.

Diagrams *A*, *B*, and *C* (Fig. 278) show some common methods of making suction connections which are practically certain to result in unsatisfactory operation, together with correct methods of making similar connections. Diagram *D* shows (in plan) another common fault in suction piping; in such a case more of the water enters the impeller on one side than on the other, and the result will be unbalanced thrust, with the likelihood of overheated bearings.

Sewage pumps are not usually fitted with foot valves because of the solid matter in the sewage. When the variable flow of sewage involves frequent starting and stopping of the pumps, water-pressure eductor primers or steam pressure exhausting primers are not used because they are liable to be wasteful and slow in operation. A vacuum pump may be installed to exhaust the air from the suction piping and pumps

when the pumps are located so as not to be self-priming. These vacuum priming units are usually selected so that they will not be seriously injured if accidentally flooded by sewage and are also installed with protective devices which will shut them down or otherwise prevent them from being flooded with sewage.

For larger stations, this may include a vacuum manifold system on which a vacuum is maintained by the priming pumps whenever the station is in operation.

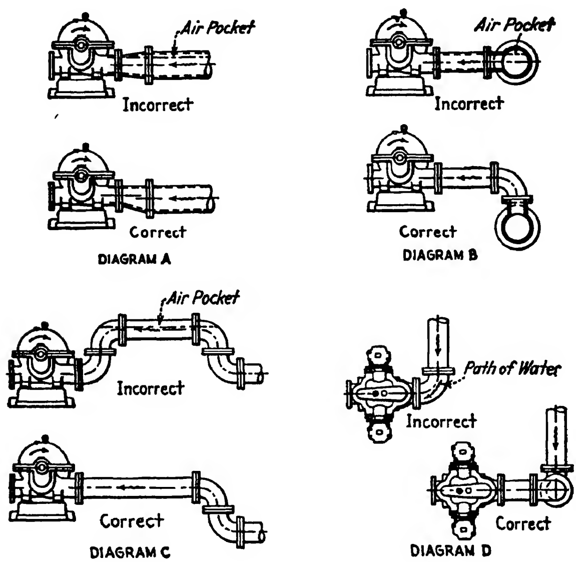


FIG. 278.—Correct and incorrect methods of arranging suction piping of centrifugal pumps. (*Dayton-Dowd Co.*)

If this manifold is at an elevation above the highest pump-suction level, all of the pumps can be kept primed all the time.

When pumps are installed without foot valves, care must be taken to prevent backflow when a pump stops, especially when the pumps may be stopped by emergency action of some protective device. This protection may take the form of check valves in the discharge pipes, sometimes automatically opened when the pump motor reaches full running operation and automatically closed by shutting down of the motor. On the larger units, this protection takes the form of a loop or siphon carried to a higher elevation than the highest discharge level. When these discharge siphons are installed they must be fitted with vacuum breakers which will automatically vent the top of the pipe to

"break" the siphon when the pump is not in operation to stop back flow through the pump due to siphonic action.

Setting Centrifugal Pumps.—It is common practice to set vertical pumps below the level of the sewage in the suction well, thus avoiding the necessity of priming. The motor is located on the main floor, above the highest water level. If the shaft is properly lined up and if adequate "steady bearings" are provided to prevent vibration in the shaft, such a method of operation is advantageous. The pump may be submerged in the suction well, but this is generally undesirable, as it is not readily accessible for cleaning or repairs. It is generally more advantageous to locate the pump in a dry well adjoining the suction well, with which it is connected by a short pipe.

Horizontal centrifugal pumps can be similarly set below the water level in a dry well, but this is usually disadvantageous because of possible damage to the motor in case of leakage of water into the dry well, as well as the greater difficulty attending satisfactory operation of a motor or engine in a deep pit. It is more common to locate the pump above high-water level, perhaps over the suction well, and provide the necessary means of priming the pump. Automatic operation of pumps in such a location is more difficult than when they are under pressure or submerged.

Characteristics of Centrifugal Pumps.—It is customary to show the characteristics of any centrifugal pump by a set of *characteristic curves* which show the relationship between head pumped against, capacity, speed, efficiency, and power required (brake horsepower). Since centrifugal pumps are usually operated at constant speed, it is common to make up a set of curves showing the relationship between the other factors for the standard speed of the pump under consideration. Thus Fig. 279 shows for a 20-in. Worthington trash pump, at 575 r.p.m., that, if the total head were 54 ft., the discharge would be 5,000 gal. per minute and the efficiency would be 56½ per cent, requiring 120 hp.; while if the total head were reduced to 34 ft. the pump would discharge 12,500 gal. per minute, and the efficiency would be 76 per cent, requiring 140 b. hp. The maximum efficiency at this speed is 78 per cent. which corresponds to discharges ranging between 10,000 and 11,500 gal. per minute, the corresponding heads being between 43 and 39 ft.

These curves also illustrate one of the very important properties of a centrifugal pump, namely, that only a certain amount of pressure can be generated if the discharge be entirely stopped, as by closing a valve upon the discharge pipe. With this particular pump, if the outlet valve be closed (and there be no discharge) the total head (lift) will rise to only 61 ft., so there will be no danger of bursting the pump or pipes. This point is known as the "shutoff head" of the pump. Moreover, under such conditions the power required will be cut to 96

hp., which is less than at any other point in the scale, so that an overloaded and burned-out motor is also impossible from pumping against a closed valve. The power would be expended in churning the water around within the pump casing, thereby generating some heat. Because of the possibility that under some conditions, if this were continued long enough, steam might be formed, causing the pump to lose its priming or causing damage to the bearings and wearing rings (if they are designed to be cooled and lubricated by the liquid flow), pumps are frequently fitted with special water piping connections to provide lubri-

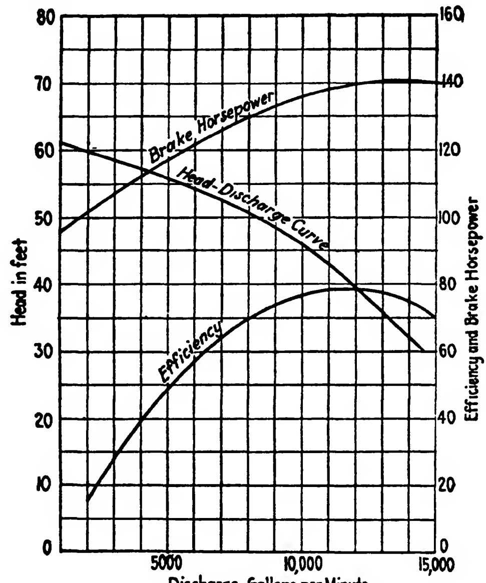


FIG. 279.—Characteristic curves of Worthington 20-inch vertical volute trash pump.

cation, whether the discharge valve is shut or not. These water connections also provide lubrication when pumps are started before they are primed, as is occasionally done with very large units, to decrease the heavier current inrush resulting from starting the motors with the pumps primed.

A very important feature of the centrifugal pump's characteristic (for constant speed) is that the brake horsepower required by the pump usually increases as the head decreases, at least for quite a range. This is due to the greatly increased quantity handled by the pump at the lower head, sometimes at considerably less than maximum efficiency. For this reason, it is important to carry out the horsepower curve beyond

its maximum point in order to know what load might be put on the motor by a broken discharge pipe or by an unusually high suction level. Commonly used designs of sewage pumps will show a maximum horsepower not much greater than the horsepower required at the best operating point and it is customary to put on a motor large enough to take care of this maximum horsepower. This is very important in sewage pumps because the time of high suction level (resulting in low total head) is liable to be the time of storm or other emergency condition when interruption of operation is most serious.

SCREW PUMPS

The impeller of a screw pump is similar in form to the propeller of a ship. Its rotation causes the water to move axially or along the shaft. Such an impeller may be installed in a casing or shell which is practically a section of pipe. Figure 280 shows a simple form of screw-pump impeller (for a small pump).

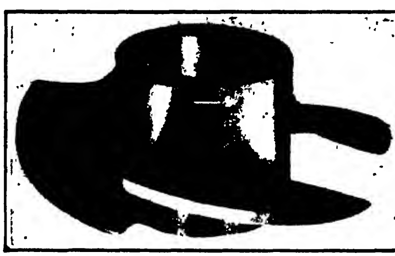


FIG. 280.—Typical impeller for small screw pump. (*Worthington.*)

The screw pump is most advantageous in handling large quantities of water when the lift is small, not over 15 or 18 ft. Its principal application has been for handling storm water which cannot be discharged directly by gravity, as at New Orleans.

The Wood screw pump, designed by A. B. Wood, mechanical engineer for the Sewerage and Water Board of New Orleans, has proved satisfactory and efficient, and has been extensively used. It is made in sizes from 3 to 12 ft. diameter. Figure 281 is a general view of such a pump. The pump proper is placed at the summit of a siphon. The legs of the siphon are essential parts of the Wood pump. They are so tapered and given such easy curves that losses of head are minimized. Tests of a 12-ft. pump at New Orleans showed discharges ranging from 600 cu. ft. per second under a head of 3 ft., to 400 cu. ft. per second under a head of 10 ft., with efficiencies above 70 per cent throughout this range, and reaching a maximum of 80 per cent. The pump was operated at a constant speed. Figure 282 shows the method of setting this pump.

The Worthington Pump and Manufacturing Company makes a pump for similar service, the general arrangement of which is shown in Fig. 283.

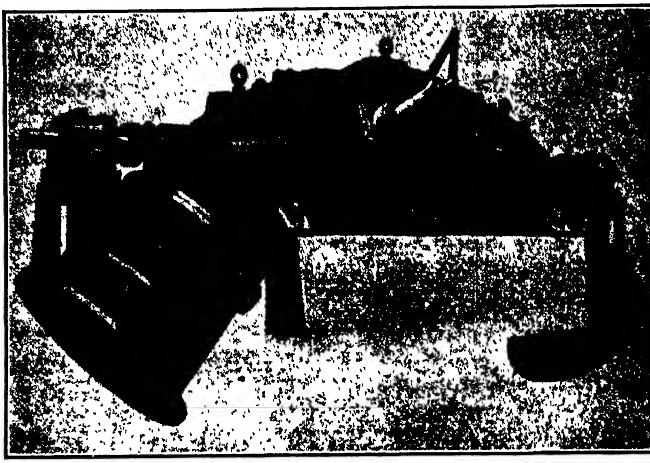


Fig. 281.—Wood screw pump.

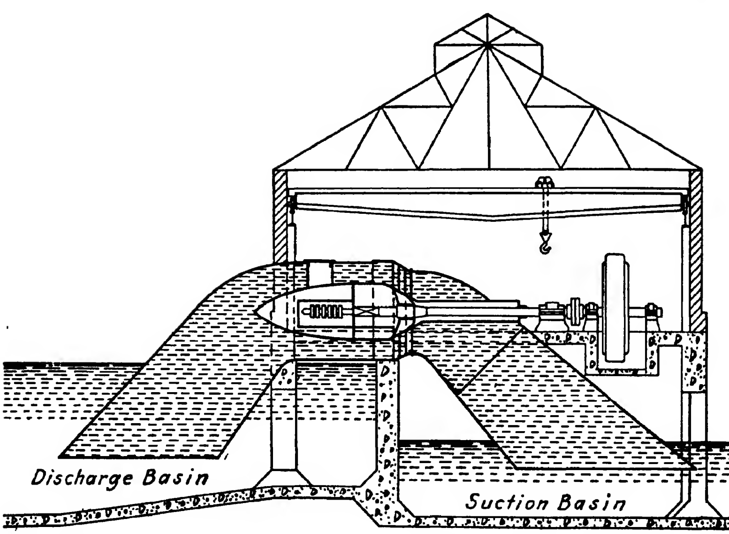


Fig. 282.—Method of setting 12-foot Wood screw pump at New Orleans, La.

This is not a true screw pump, although called by that name; but rather a combination of a screw and centrifugal pump.

The screw pump usually has the very desirable characteristic that with decreased head the capacity increases without correspondingly increasing the power required by the pump after passing the best operating point. Therefore, with high water on the suction side or low overall head, the pump is more dependable rather than less dependable. This feature is more important in a station containing several units, or when several pumps receive their power from the same source of energy.

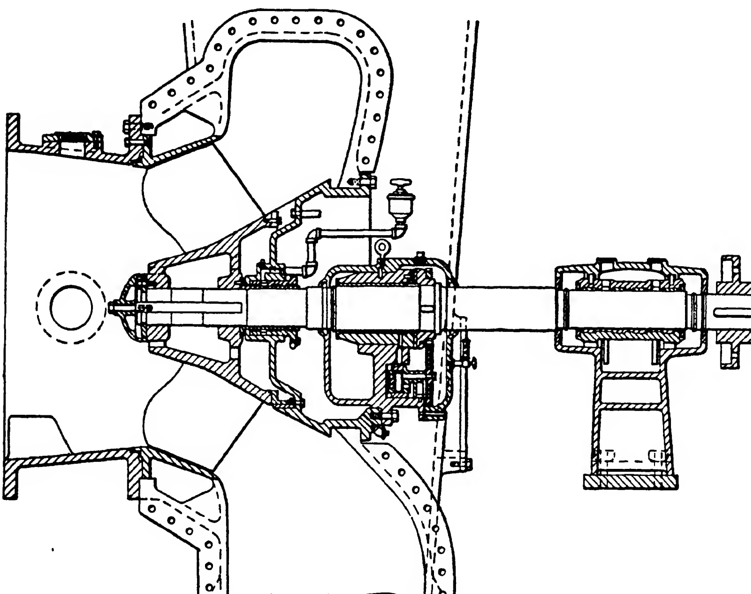


Fig. 283.—Cross-section showing general construction of Worthington screw pump for drainage and irrigation work.

PNEUMATIC EJECTORS

A pneumatic sewage ejector consists of an air-tight iron receiver with check valves on the inlet and outlet pipes, provided with a float-controlled valve or switch by the action of which compressed air is admitted to the receiver when the latter is filled.

In municipal sewerage works, ejectors are sometimes used to lift the sewage from small districts which are too low to be drained by gravity; and they have also been used extensively for pumping sewage from deep basements into city sewers.

One of the earliest types of pneumatic ejector is the Shone, the present form of which is illustrated in Fig. 284. This ejector is now made by Yeomans Bros. Co. Its operation is extremely simple. Sewage enters

the receiver through the check valve *B*, and accumulates until the entrapped air lifts the inverted cup *D*, which closes the exhaust valves and opens the compressed air valve. The air then flows into the receiver and forces the sewage through the check valve *F* and into the discharge pipe, until the level of the liquid falls to such a point that the weight of cup *C* and the water it contains causes the cups to drop and reverses the air valves. While the ejector is discharging any flow of sewage is retained in the inlet pipe, above check valve *B*. Figure 284 shows the

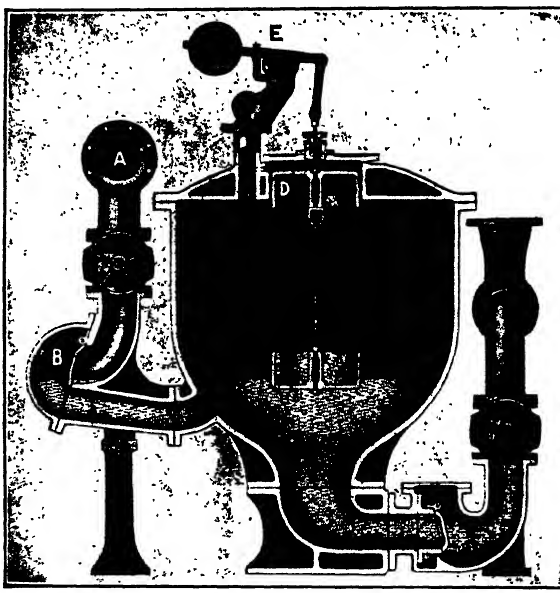


FIG. 284.—Shone sewage ejector.

ejector discharging but nearly empty. The operation will be repeated as long as sewage flows to the receiver and the supply of compressed air is maintained, and the seating of the check valves is not prevented by solid matter.

These units discharge intermittently and consequently are usually installed in pairs and interlocked so that one discharges while the other is filling. This gives a more nearly uniform rate of flow in the discharge line and permits using a smaller discharge line than would be required if the pipe line had to handle the whole flow in intermittent periods of half the total duration.

The Jennings ejector (Nash Engineering Company) is similar in principle to the Shone. The principal differences in construction are in the

details of the check valves and float; but the float operates an electric switch controlling an electrically driven air compressor, located in the same chamber as the ejector proper. This compressor, the "Nash Hytor," is, therefore, an essential part of the apparatus. Figure 285 illustrates the Jennings ejector.

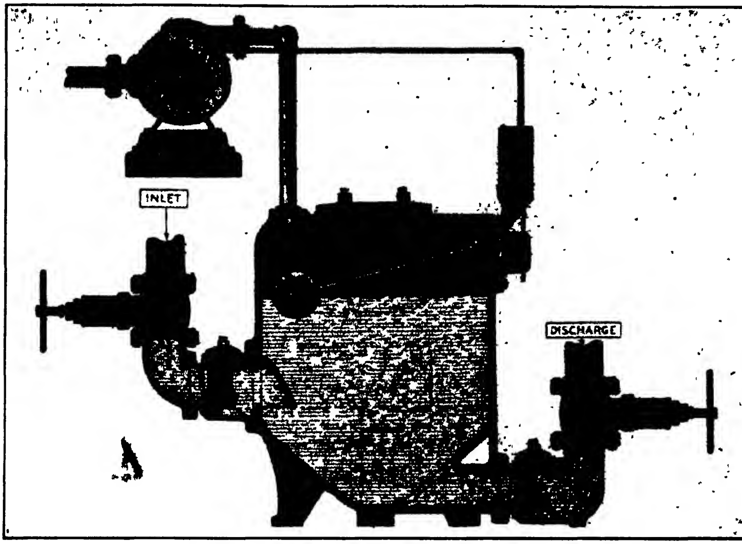


Fig. 285.—The Jennings sewage ejector.

EJECTORS USING CENTRIFUGAL PUMPS

The term "sewage ejector" is not limited to pneumatic ejectors, but is used to include also any type of apparatus suitable for the same service. "Electric centrifugal" ejectors are in common use. They are in effect small automatic pumping plants, including receiving tank, vertical centrifugal pumps, electric motors, and float-controlled switches. Figures 286 and 287 illustrate typical ejectors of this kind.

The Yeomans ejector (Fig. 286) uses a pump having a nonclogging impeller, and no screens are provided. The Chicago Pump Company claims particular advantages for its "Flush-Kleen" ejector (Fig. 287) because of the interposition of a strainer, as shown. All sewage entering the receiver passes through this strainer and then flows through the pump to reach the receiver. When the pump is started the discharge is through the strainer in the reverse direction, thus providing for automatic cleaning of the screen. The ejector illustrated is designed for installation in a dry well beside the receiver, but the same principle is applied in a submerged ejector for installation in the collecting well.

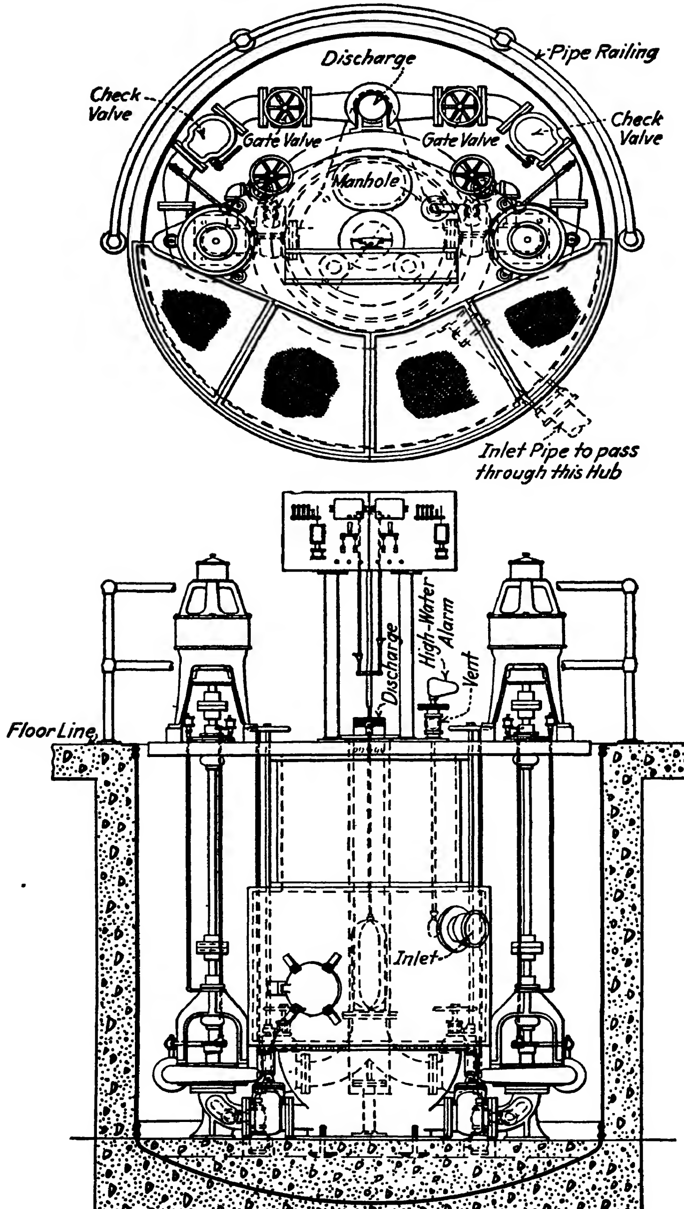


FIG. 286.—Yeomans duplex ejector.

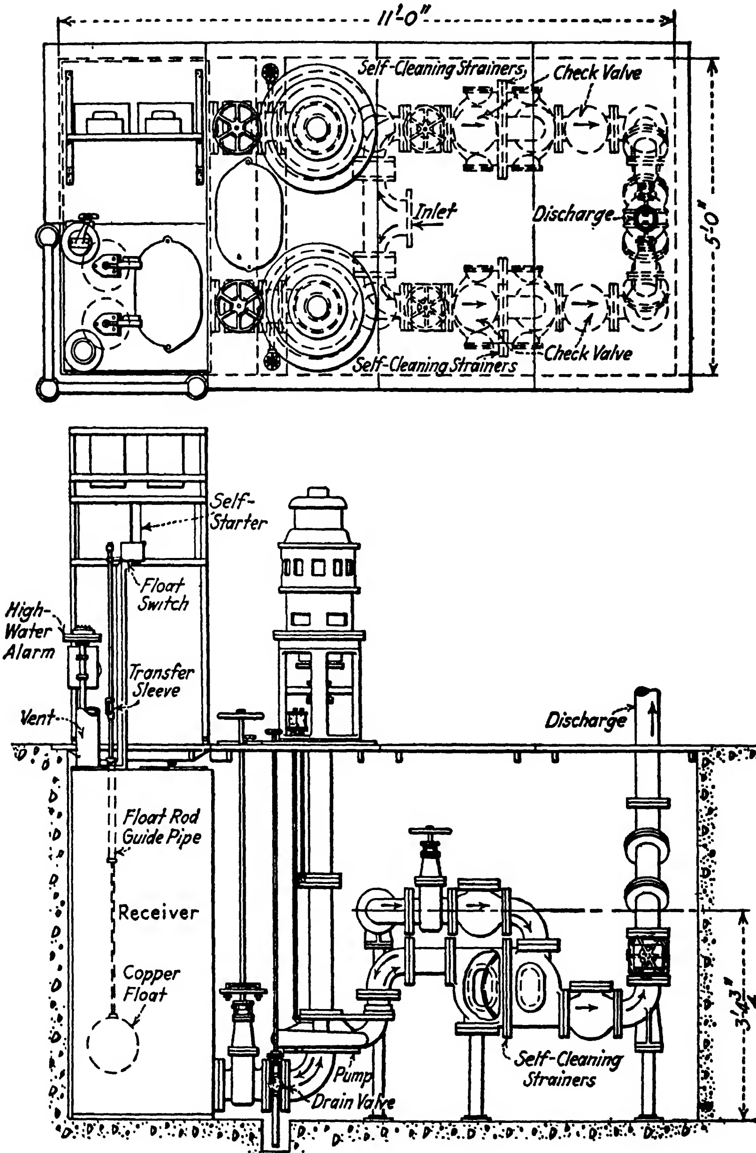


FIG. 287.—"Flush-kleen" dry basin sewage ejector. (*Chicago Pump Co.*)

The same company also makes an ejector in which an ordinary pump is suspended in the sewage in the collecting well, and a screen basket is used over the inlet pipe; and another in which their "nonclog" pump is suspended in the well, and screens are considered unnecessary.

Centrifugal pump ejectors may be employed to lift the sewage of small districts which are too low to drain by gravity. Although there is no clearly defined line of demarkation, the smaller installations are likely to be called ejectors, and the larger, underground pumping stations. Figures 288 and 289 show two typical forms of such pumping

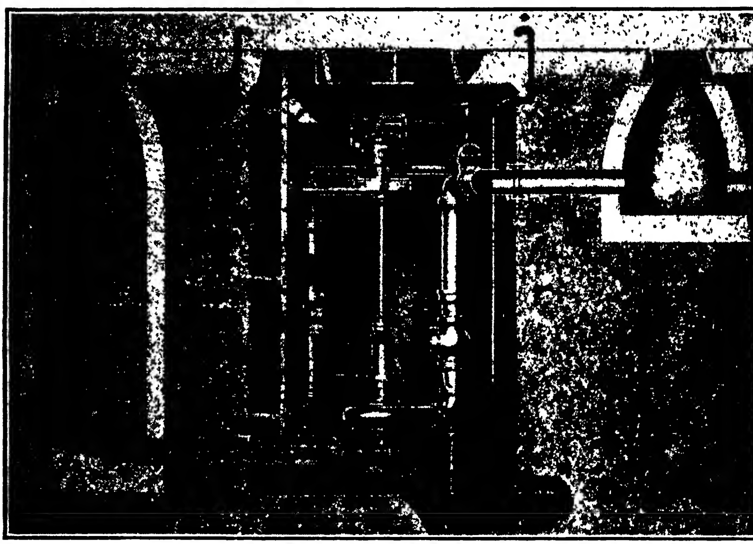


FIG. 288.—Fairbanks-Morse underground steel tank sewage relift station.

stations in steel tanks, as made up by pump manufacturers. The tanks are designed to be watertight, so that there should be no water in them except perhaps from condensation of moisture from the air. The manhole covers, however, cannot be depended on to be tight, and in case of very heavy rain when the gutters were flooded, sufficient water might enter the chamber to fill it and injure the motor. The Yeomans flood-proof station shown in Fig. 289 would be safe in such a contingency, since the entrance tube forms a trap which would retain air in the top of the chamber, so that water could not get to the motor.

DESIGN OF PUMPING PLANTS

In addition to the selection of type of pump and prime mover, the design of a pumping plant for sewage requires the detailed study of

several matters of major importance, as well as many minor items. Among the most important items are amount and variation of sewage flow, suction well, racks or screens, grit chamber, lift of pumps (including friction in pipes), number and size of pumps, reserve pumps, source

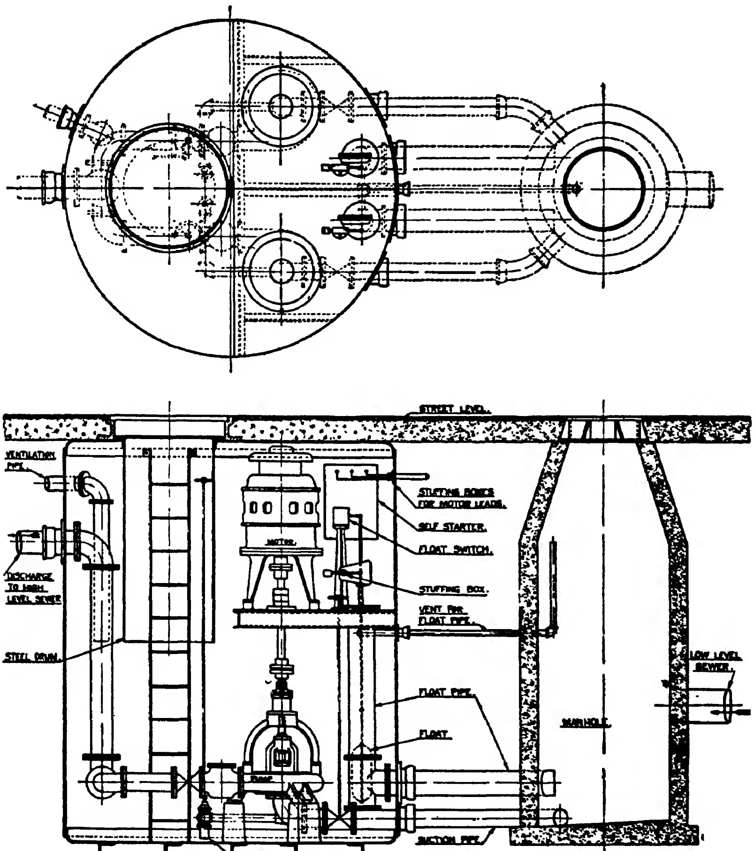


Fig. 289.—Yeomans flood-proof underground sewage pumping station.

of power, supplementary power, method of starting and stopping pumps, size and location of piping and valves, meters, and gages.

Suction Well.—The rate of sewage flow usually fluctuates widely from time to time during the day, and also varies with the day of the week and with weather conditions. In order to compensate for such fluctuations and make it possible for pumps to run at nearly uniform rates, without frequent stopping and starting, it was formerly the custom to

provide a suction well, with storage capacity sufficient in some cases to retain several hours' flow. Improvements in electrical apparatus, making it possible to stop and start pumps automatically as often as conditions make it desirable, and the increasing popularity of electrically driven pumps, have made it practicable to operate sewage pumping plants with little or no storage; and as detention of sewage as the pumping station for sufficient time to allow it to become septic, or to permit appreciable sedimentation, is undesirable, suction wells are now commonly made small or entirely omitted.

When used, a basin of considerable area and small depth is generally to be preferred, since the increase in lift corresponding to drawing down the

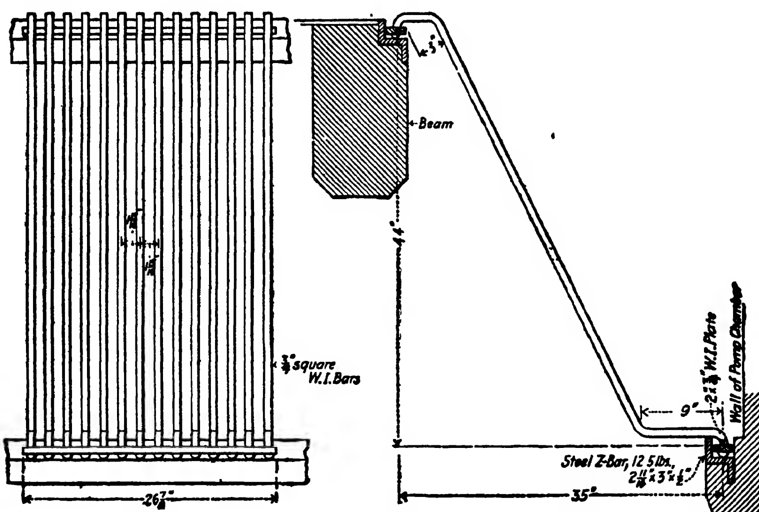


FIG. 290.—Rack used at North Attleborough pumping station. (*Barbour.*)

sewage in the suction well may be a material proportion of the total lift, and the operation and efficiency of a centrifugal pump may be seriously affected by such a change.

Racks.—The subject of racks and screens is treated in considerable detail in Vol. III, Disposal of Sewage, with particular reference to their use as elements of a sewage treatment plant.

Racks of some type are generally desirable, and sometimes necessary, in connection with sewage pumps, to prevent clogging the pumps by solids. Where nonclogging centrifugal pumps are used, the necessity for racks is not as great as though reciprocating pumps or centrifugal pumps of the ordinary type were used; but the smaller sizes of even nonclogging pumps are certainly unable to pass solids frequently

found in sewers, and it is generally wise to provide coarse racks to protect large pumps.

Where racks of any type are provided, it is essential to arrange for their regular and frequent cleaning. Racks of inclined bars with clear spaces not exceeding 2 to 2½ in. are to be preferred. For very small plants a type of rack which will not cause trouble from clogging, even if it should be without attention for several days, is desirable.

Figure 290 shows a type of rack which is distinctly advantageous for such conditions. It was designed by Frank A. Barbour and has been used by him at North Attleborough, Marblehead, and other places. Located opposite the end of the sewer, and of considerable length compared to the diameter of the sewer, the rack is unlikely to clog sufficiently to cause trouble. It is easily raked by hand from a platform at the upper edge of the rack, and sections are easily removed and replaced when necessary. The cost of construction is not excessive.

Head on Pumps.—Attention has already been called to the fact that changes in lift resulting from fluctuating water level in suction wells may be of significance in the case of centrifugal pumps. If the sewage is discharged through a force main of considerable length, changes of the frictional resistance corresponding to changes in rate of discharge may be of much greater significance, particularly if there are several pumps and the number of pumps which may be in simultaneous operation is varied. It is important that pumps be selected which can operate satisfactorily throughout the range of lifts which may be experienced.

Reserve Pumps and Supplementary Power.—Even in small plants it is generally advisable to provide at least one reserve pump for use in case of accident or repairs. In steam-driven plants, there should also be at least one boiler in reserve. When power is provided by internal combustion engines, at least one engine should be provided as a reserve source of power. Such reserve engine may be of a type less expensive in first cost but more costly in operation than the engines in regular use.

Supplementary power, from a different source than that generally employed, is desirable when practicable. Electricity is not generally practicable for supplementary power, on account of the high standby charges; but when practicable, electric power from other sources than those regularly used is extremely advantageous. Steam is undesirable, even when an old steam plant is to be superseded, because of the rapid deterioration of unused boilers, and of the time required to get up steam. Internal combustion engines are generally the most practicable source of supplementary power. In case of a small plant using horizontal centrifugal pumps, a gasoline engine at the opposite end of the pump shaft from the motor may be possible, disconnecting the coupling to the motor and connecting that to the engine when the latter is to be used. In the case of vertical electric-motor-driven pumps, the only practicable source

of supplementary power is likely to be an electric generator within the pumping station, driven by an internal-combustion engine.

Automatic Control.—Electrically driven pumps may be stopped and started automatically by float-controlled switches. The development of this apparatus has been such that almost any desired operating schedule can be accomplished automatically, the pumps coming into operation successively in any desired order, and in the case of variable speed pumps, being speeded up or slowed down in accordance with changes in the rate of sewage flow or elevation of water in the pump well. An excellent example of such control is the Westerlo Station at Albany, described in subsequent pages.

Variation from the best operating speed of a motor-driven centrifugal pump is liable to considerably reduce the overall efficiency of the unit because of the reduced efficiency of the motor combined with the reduced efficiency of the pump at other than the best speed of both. When lack of storage capacity makes it necessary to follow variations in flow fairly closely, it is desirable to have several constant-speed units operated at their best speed, with small variations in flow cared for by variation of speed of only one unit. Large variations in flow are cared for by starting and stopping one or more of the constant-speed units.

Electric power for pumps involves the usual forms of protection for the motor itself against overload, under-voltage and other irregularities of operation, according to the size and type of motor and the characteristics of the power system from which it is supplied with energy.

With the increasing use of electric motor drive for pumps, much attention has been given to the automatic control of the motors. It is considered good practice to provide at least for handling the starting and stopping of the motors automatically after starting has been initiated by the pump attendant.

Occasionally this is carried to the point of full automatic control of the pumping. When this is done, the ordinary electric protective and control equipment is extended to perform the following functions:

Start the pump at high suction level.

Stop it at low suction level, usually by float control.

Go through the priming process in starting.

Protect the pump while in operation, against damage because of air leaks, loss of priming, plugged suction pipe or pump, overheated bearings, broken discharge line.

When a pumping-station control is made fully automatic, it is necessary to introduce features which will cut off a pump permanently from the source of energy after it has made a certain number of attempts to start without coming successfully into full operation, with provision for an alarm notifying a responsible attendant of the failure of the equipment to function.

PUMPING STATIONS

Pumping stations have been classified in a variety of ways, such as according to capacity or nature of prime movers, but there is nothing gained by such an artificial analysis. The authors have accordingly prepared brief descriptions of a number of stations, which illustrate the great variety of ways in which the problems due to poor foundations, variable capacity requirements, and different methods of obtaining power have been solved. In some cases, details have doubtless been employed which were due to local conditions and would not be selected for a standard design; in studying the various plans, particularly the type of pump drive, this influence of local conditions should not be overlooked.

Ward Street Pumping Station, Boston (Metropolitan Sewerage Works).—This station, as originally constructed, is typical of the best practice of its date, where high-duty pumping engines were employed. It was built in 1902–1904. The buildings are of brick with granite trimmings, and include an engine room 65 by 120 ft. for three 50,000,000-gal. pumping engines, of which two were provided; a boiler room 38 by 105 ft., for four vertical boilers; a coal pocket 39 by 52 ft.; a screen chamber[1] 42 by 47 ft.; and a chimney 150 ft. high with a 6-ft. flue. Fig. 291 shows plan and section of the station. The pumping engines (Fig. 292) were built by the Allis-Chalmers Company, and included vertical triple-expansion steam engines with cylinders 21, 38, and 58 in. in diameter, with stroke of 60 in.; and there are three outside packed single acting plungers 48¼ in. diameter and 60 in. stroke. The fly wheels, two for each engine, are 18 ft. in diameter, and the speed is 25 r. p. m. for a capacity of 50,000,000 gal. per day. The lift is 45 ft. Each suction and delivery chamber contains 36 valves, making a total of 432 valves for each engine. The nominal area of the waterway through the suction and discharge valves is about 200 per cent of the area of the pump plungers. Upon duty trial in 1906, these engines developed a duty of 152,700,000 ft.-lb. per 1,000 lb. of dry steam. Steam pressure was 150 lb. per square inch and slip was 3 to 3.6 per cent.

Although these pumping engines have proved satisfactory and are still in use, nevertheless, when a third pump was required, instead of providing another of the same type (as contemplated in the original plans), a centrifugal pump was selected.

The station was originally put in service in 1904. Since that time the design of centrifugal pumps has been so improved that it has been considered best to substitute one of this type for the plunger pump originally intended. The cost of the centrifugal unit as compared with the plunger type unit is at present probably in the ratio of about 1 to 6.[2]

[1] The screens at this station are described in "American Sewage Practice," Vol. III, First Edition, 321 *et seq.*

[2] *Report* of Metropolitan District Commission, 1924.

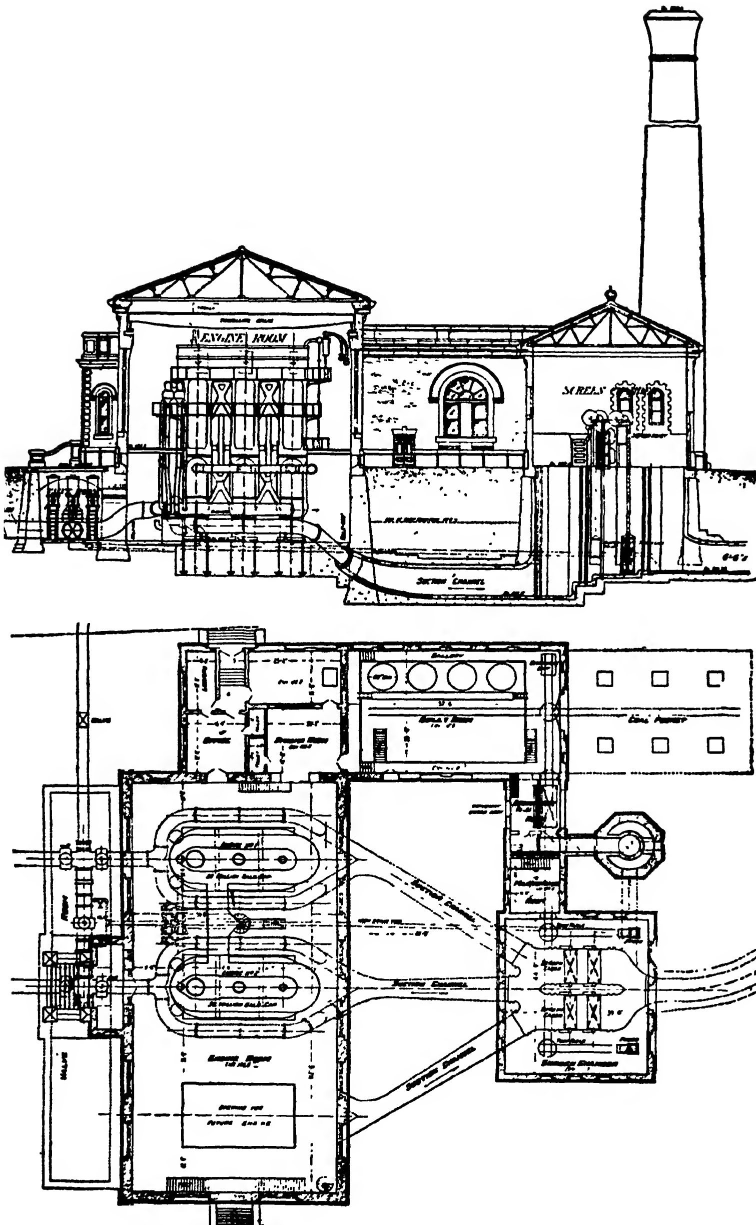


Fig. 291.—Ward Street pumping station as originally built (Boston, Mass.).

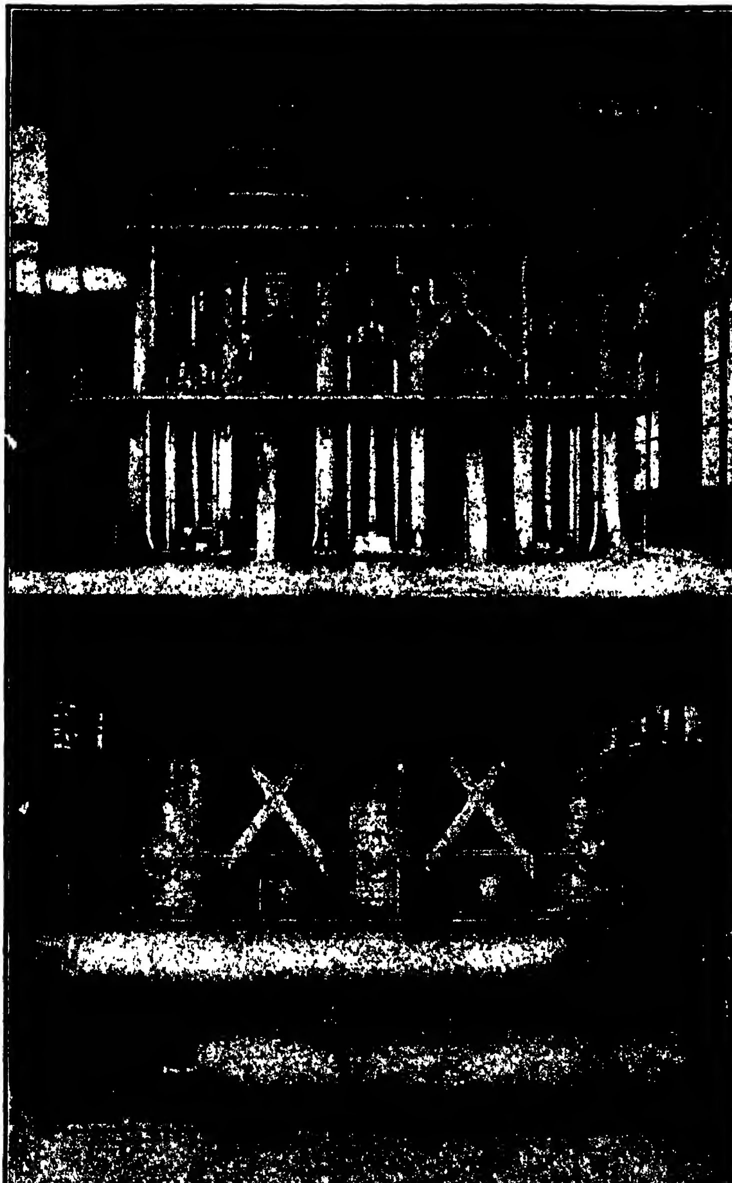


FIG. 292.—50 mil. gal. pumping engine at Ward Street pumping station.

The unit selected consists of a 36-in. Morris Machine Works centrifugal pump with a capacity of 50,000,000 gal. per day against a total lift of 45 ft. (of which 14 ft. are suction lift), directly connected to a Nordberg uniflow steam engine of 540 hp., and running at 150 r.p.m., with a guaranteed duty of 103,000,000 ft.-lb. per 1,000 lb. of dry steam.

Baltimore.—The Baltimore sewage pumping station is provided with a main engine room 182 ft. long, 50 ft. wide, and 68 ft. high from the basement floor to the chords of the trusses. It was planned to contain eventually five pumping engines, two drainage pumps, a 20-ton electric crane, an electric switchboard, and valves and piping. Three pumping engines were installed in 1906, which were built by the Bethlehem Steel Company. These are of the vertical, triple-expansion, crank and flywheel type (Fig. 293) rated at 27,500,000 gal. in 24 hours against a head of 72 ft. when the speed is 20 r.p.m. The pump has three single-acting plungers, 40¼ in. in diameter and 60 in. in stroke, and the valve chambers have very large flap valves. Each engine is rated at about 400 hp. at normal speed. On test the average duty was 165,000,000 ft.-lb. per 1,000 lb. of dry steam, with an average slip of about 3¾ per cent. Under operating conditions the slip has varied between 6.5 and 10.8 per cent during the years 1914–1923, and the average quantity pumped from 18 to 28 million gallons daily.[1] The discharge is through two 42-in. cast-iron force mains, each provided with a 42- by 21-in. Venturi meter. Between the engine room and the boiler room is a screen chamber where the sewage is first sent through movable screens and then through finer fixed screens over the ends of the suction pipes. The movable screens are in the form of bar cages, of which there are two in series in each of two channels. Each cage is 8 ft. 1 in. high, 7 ft. 6 in. wide, and 3 ft. 3 in. deep. The bars are ¾ in. in diameter and are staggered in such a way as to provide clear openings 1 in. wide. The fixed screens are gratings inclined 74 deg. from the horizontal, with clear spaces ¾ in. wide.

When preliminary designs were made of the station in 1906, centrifugal pumps had not reached their present stage of perfection and for that reason they were not used even though the first cost would have been considerably less. It is very likely, however, that the pumps which will be installed in the future will be centrifugals.[1]

Columbus, Ohio.—A sewage pumping plant built at Columbus, Ohio, from the plans of John H. Gregory[2] is shown in Fig. 294. The engineer's description of it is as follows:

The sewage is first admitted to a long chamber, serving as a sandcatcher, is screened to remove the coarser matters in suspension, and then passes into the suction well. The screening device consists of two cages of steel-

[1] C. E. Keefer, *Public Works*, 1924; **55**, 207.

[2] *Trans. Am. Soc. C. E.*, 1910; **67**, 282.

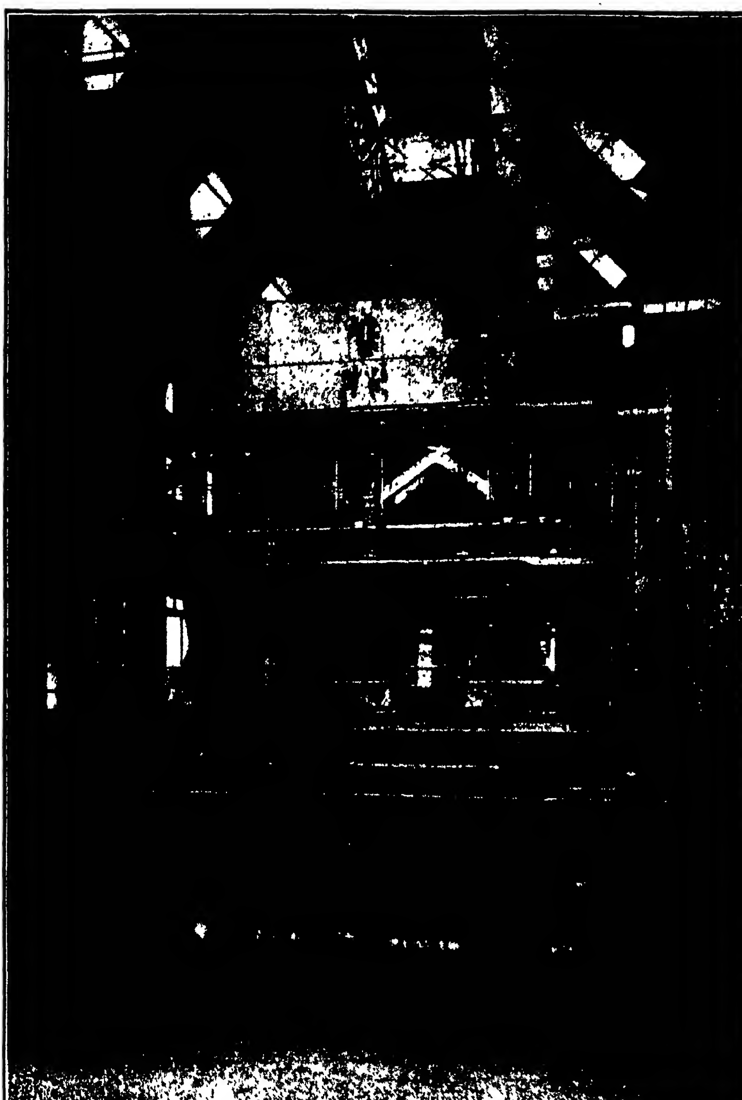


Fig. 293.—Interior of Baltimore pumping station.

frame construction, holding removable sets of screens made up of ¾-in. square bars, 1 in. apart in the clear. The cages are raised and lowered by hand by a movable screen lifter hung from a traveling hoist and runway just below the ceiling of the screen room above. The sub-structure is of concrete, reinforced at various points. In the sub-structure of the engine room, in which are located the pumps and engines on account of the suction lift, the walls are lined with hard vitrified red pressed brick.

The walls of the superstructure are of brick, faced with red pressed brick outside. In the engine room the walls are lined with light buff-speckled pressed brick, and in the screen room with hard red brick. The stone trimmings are all of Bedford limestone. The ceilings in both rooms are all of plaster on metal lath, fastened to the lower chords of the roof trusses. The roof is of 3-in. hollow terra cotta tile and slate carried by steel trusses and intermediate framing.

The pumping machinery is installed in duplicate. Each unit consists of a Columbus, horizontal, four-stroke-cycle gas engine connected by a Morse silent-running high-speed chain to a horizontal, single-stage Worthington volute pump with 12-in. suction and 10-in. discharge nozzles. The engine is capable of developing 90 hp. when operating on natural gas having a thermal efficiency of about 1,000 B.t.u. per cubic foot. When running together, each unit has a rated capacity of 2,200,000 gal. per 24 hours against a head of 75 ft., and when running alone a maximum capacity of 2,900,000 gal. per 24 hours against a head of 63 ft. For starting the engines, the equipment includes a small motor-driven air compressor and air tank.

The sewage is pumped through a 20-in. cast-iron force main to a point about 8,180 ft. from the pumping station, where it is discharged into the upper end of the Mound St. sewer. The flow is measured by a 20-in. Venturi meter, the register, chart recorder, and manometer being placed in the pumping station. The meter tube is of special construction, and between the tube and the register and manometer, oil seals are interposed to keep the sewage out of the latter.

Chicago, Ill. (Calumet Pumping Station).[1]—This station, built by the Sanitary District of Chicago in 1920–1921, is an example of a large station designed to handle storm water as well as sewage, with discharge into either of two outlet conduits at different elevations; large suction and discharge basins; mechanically operated rakes for clearing the bar screens; two groups of three pumps each, one set of comparatively small size (36 in.) for dry weather, the other of large size (72 in.) for storm flows; a standby source of power from generators driven by Diesel engines (in a separate building nearby), and two motors of different sizes on opposite ends of two of the largest pumps.

The general arrangement of the inlet and discharge sewer, screen chamber, suction chamber, and discharge basins, is shown in Fig. 295, and the layout of the station itself in Fig. 297. Each of the three smaller pumps has a capacity of 50 to 75 cu. ft. per second, depending on

[1] Eng. News-Record, 1920; 85, 872: 1921; 86, 65.

the lift, and each of the larger pumps, 200 to 275 cu. ft. per second. The lower ends of the suction and discharge pipes are submerged in the suction and discharge basins, and the highest point of the invert of the discharge pipe is above high-water elevation in the discharge basin. This arrangement makes it unnecessary to use check valves, which are both costly and unsatisfactory in such large sizes. Gate valves are provided on the smaller pumps for throttling purposes.

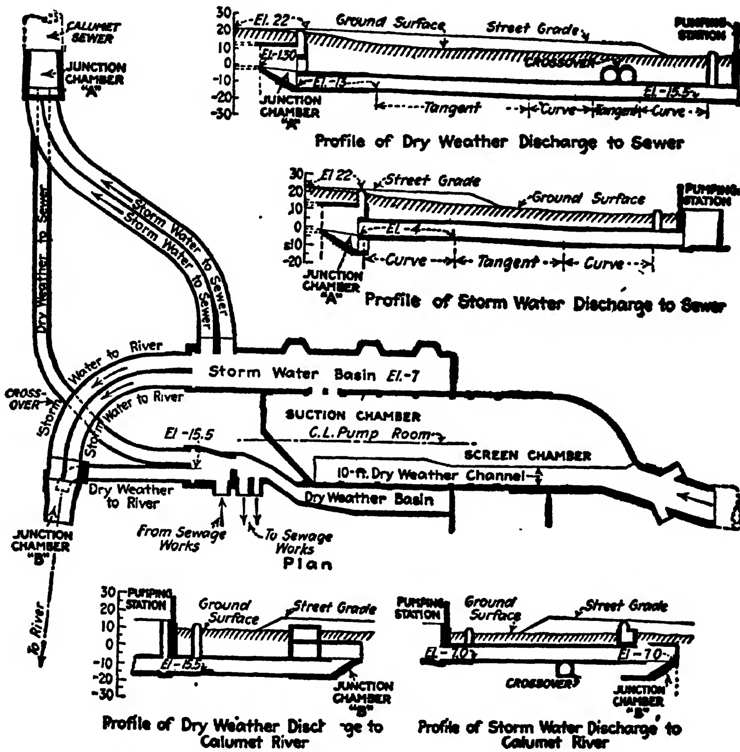


FIG. 295.—General arrangement of conduits and suction and discharge basins, Calumet pumping station, Chicago, Ill.

The racks are built of 4- by ½-in. bars, set at an angle of 37½ deg. to the vertical, with 5 in. clear space between bars. They were provided with traveling rakes, as shown in Fig. 296. The rate of travel of the rakes is about 35 ft. per minute.

The pump motors are of the synchronous type; those for the 36-in. pumps are rated at 106 hp. and run at 257 r.p.m.; each of the 72-in. pumps has a motor of 1,100 hp. at 150 r.p.m., and two of these pumps

are also provided with 450 hp. motors at 106 r.p.m., which can be used when pumping to the low level conduit. Vacuum pumps are provided for priming and to maintain the vacuum during operation, as the suction lift will generally vary between 12 and 18 ft.

George M. Wisner was chief engineer of the Sanitary District of Chicago when this station was designed, and was succeeded by Edward J. Kelly, under whom it was constructed.

New Orleans.—The main sewage pumping station at New Orleans was built in 1902. At this point most of the sewage of the city is lifted

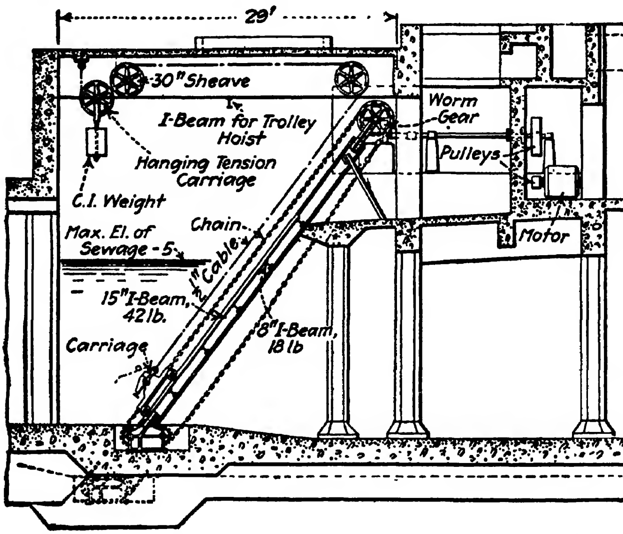


FIG. 296.—Automatic cleaner for trash rack at Calumet pumping station.

and discharged through a 48-in. force main about 7,000 ft. long into the Mississippi River. The lift varies considerably, with the elevation of water in the river, and is also affected by the difference in frictional resistance in the force main at various rates of discharge. The two original pumps were driven by compound steam engines and had capacities ranging between 37 cu. ft. per second at 54-ft. lift and 22 cu. ft. per second at 30-ft. lift, the variation being obtained by varying the speed. The impellers of the centrifugal pumps were 110 in. in diameter and had large passages, but considerable portions of the periphery were blanked off, and considerable vibration was experienced in operation. By 1913, the capacity of one pump was insufficient to take care of the sewage flow at times, and no increase in capacity could be obtained by operating both pumps when discharging through the long force main. One two-

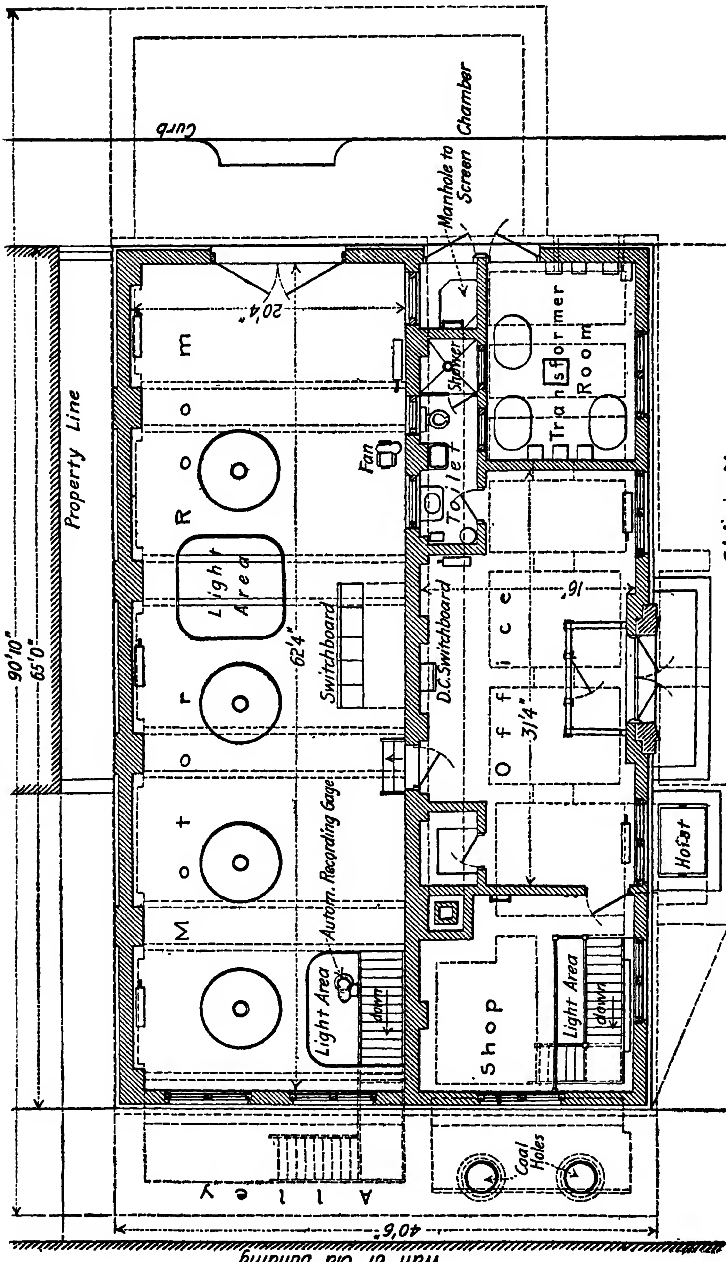


FIG. 298.—Automatic pumping station, Union Park Street, Boston, Mass.

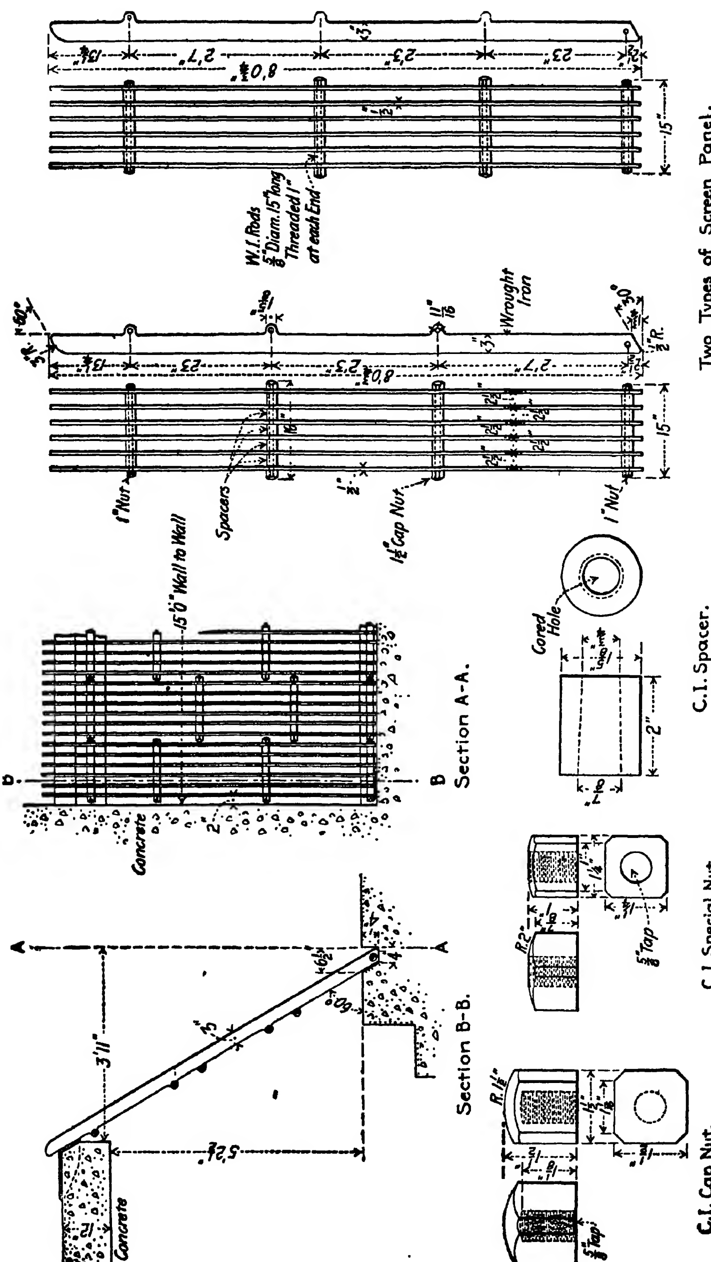
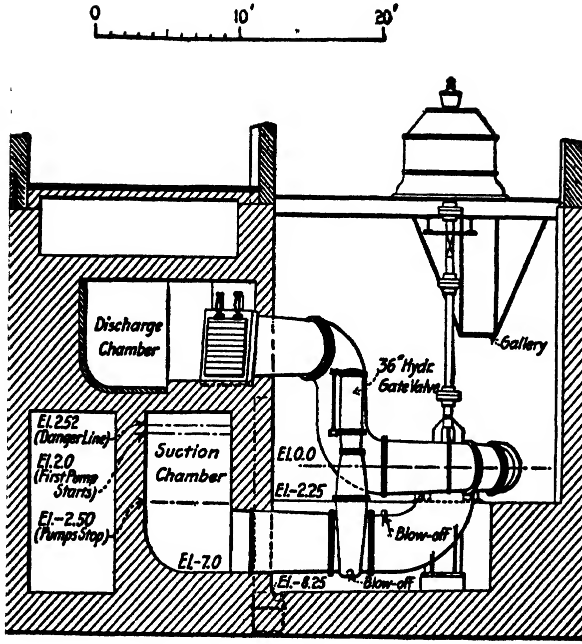
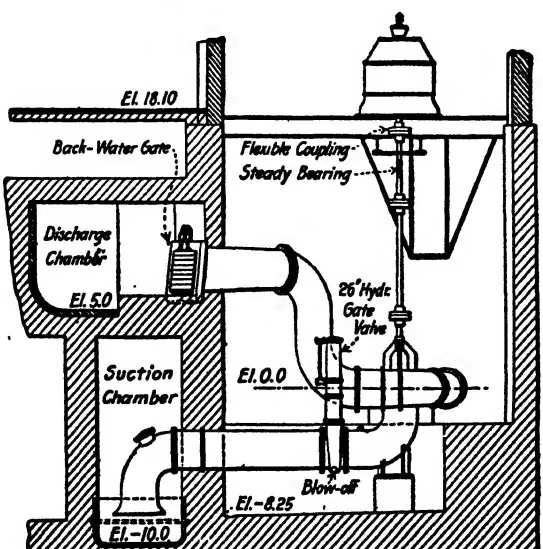


FIG. 299.—Sewage racks used at Boston, Mass.



Elevation of 36-in. Unit.



Elevation of 24-in. Unit.

(Facing page 708)

speed electric-motor-driven pump with impeller of the trash-pump type was then built for the station and was in substantially continuous operation until 1926. This pump at 248 r.p.m. discharged from 55 to 85 cu. ft. per second as the head varied from 43 to 35 ft.; and at 364 r.p.m., 106 to 133 cu. ft. per second on lifts from 87 to 72 ft. The impeller was 57 in. in diameter. In 1925 a second 48-in. force main was provided and in 1926 two 24-in. trash-pumps were installed, driven at 300 r.p.m. by synchronous electric motors. One of these pumps discharging through one force main against high water in the river will pump 95 cu. ft. per second against 70-ft. lift; and both pumps together, 120 cu. ft. per second against 90 ft. One pump operating alone against low water in the river will pump 110 cu. ft. per second against 64-ft. lift. The runner is 63 in. in diameter, and can pass solids 21 in. in diameter.

George G. Earl has been general superintendent and Chief Engineer of the Sewerage and Water Board since its establishment.

Union Park Street Pumping Station, Boston.—This station was built in 1914, from plans of C. H. Dodd, prepared under the general direction of E. S. Dorr, then engineer of the sewer division. At the time of its construction it was probably the largest existing sewage pumping station with automatic control. The superstructure is 60 by 40½ ft. in plan; the basement is much larger. Along one side of the building extends the motor room, and in order that machinery may be moved into and out of it readily there is a large doorway at one end and a return in the curbing, so that a motor truck can be backed into the building for some distance, the floor being strengthened for the purpose. A transformer room in one corner of the building can be entered only through an outside door, the keys to which are in the possession of the employees of the local electric-light company, the sewer service having no responsibility for the care and maintenance of the transformers. Adjoining the transformer room is a small room, also entered only through an outside door, affording access to a manhole leading to the screen chamber. The remainder of the ground floor is occupied by an office with a large store closet and by a shop.

The station contains three 150-hp. squirrel-cage motors, each driving a 36-in. centrifugal pump, the capacity being 30,000 gal. per minute against a lift of 13½ ft. at a speed of 194 r.p.m.; and a 75-hp. squirrel-cage motor driving a 24-in. pump with capacity of 15,000 gal. per minute against a lift of 13½ ft., at a speed of 233 r.p.m. (Figs. 298 and 300). The sewage enters the station through a screen chamber, provided with a rack constructed according to the details shown in Fig. 299. The rack is in 12 panels, each 15 in. wide, and 8 ft. ¾ in. long. The general arrangement of this screen chamber is shown in Fig. 301.

The pumps are controlled by a float in a well, one well sufficing for all pumps, the switch mechanism throwing into service one pump after

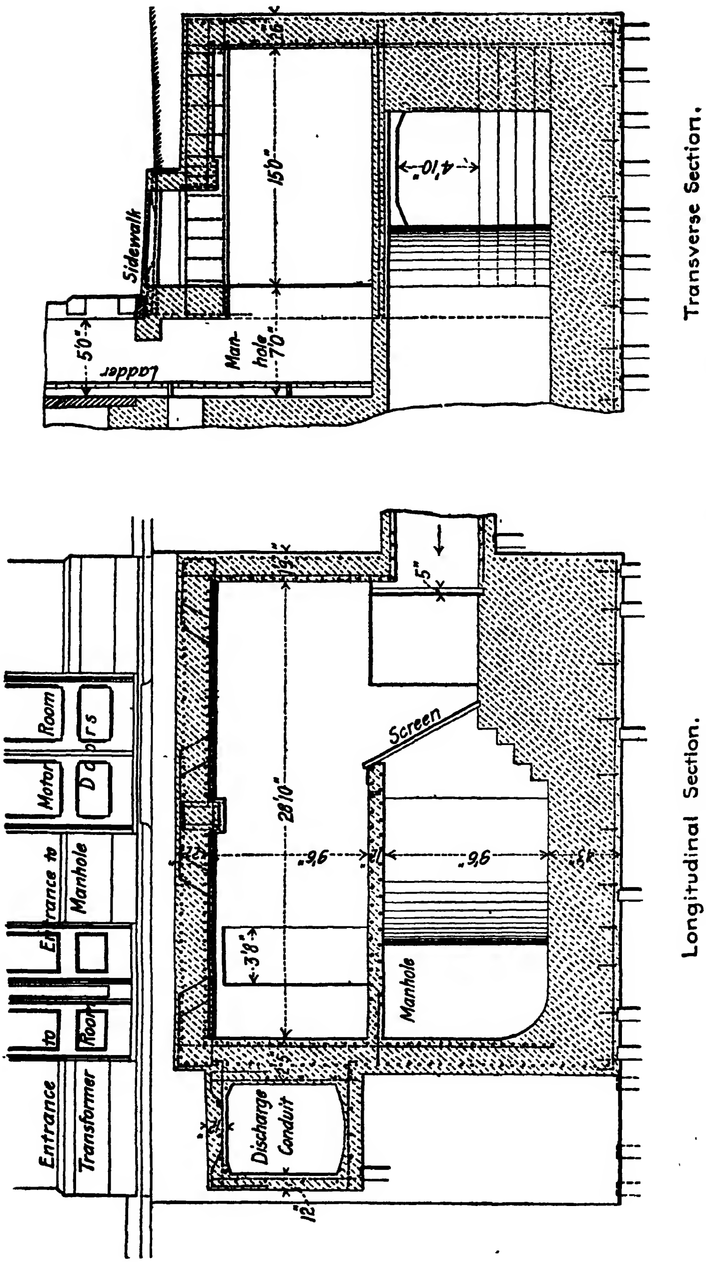


FIG. 301.—Screen chamber, automatic station, Boston, Mass.

the other as the level of the sewage in the suction chamber rises. The electric devices for this purpose were furnished by the Cutler Hammer Company. There is another float well in this station which operates an automatic recording gage of a type in use in several places on the Boston sewerage system. It has a pen moved vertically by the float rod over the surface of a chart which is revolved horizontally by clockwork.

The small pump has its suction run into a sump 3 ft. lower than the remainder of the suction chamber, so that this pump can be used to drain the station down to the level of the pump-room floor. Below that grade the drainage is removed by hydraulic eductors with suctions in small cast-iron sumps in the concrete floor.

The positions of the 2½-in. bronze nipples and gate valves for blowing off each pump casing and the bottom of each hydraulic gate valve are indicated in Fig. 300. The hydraulic gate valves are connected by 1-in. pipe with the street mains. The end of the discharge pipe has a large backwater gate.

Albany, New York.—The Westerlo sewage pumping station[1] built in 1915–1916 from plans of Hering and Gregory, is circular in plan, and has an annular suction well surrounding the dry well in which the pumps are located. It also has two sets of pumps, one set being driven by constant-speed motors, the other by variable-speed motors, thus providing for a range of discharging capacity from 10 to 65 million gallons daily. Figure 302 shows plans and vertical section of this station.

Sewage flows to the pumping station through an intercepting sewer 4 ft. 6 in. diameter, and before reaching the station, it passes through a grit chamber in which there are racks of ½- by 3-in. bars set 2½ in. apart on centers, and at an inclination of 1 vertical on 2 horizontal.

The circular form for the station was adopted largely for reasons of economy, since the bottom of the station is more than 25 ft. below normal groundwater elevation. The annular suction well also has some advantages, since the flow can be easily deflected so as to pass around the pumps in either direction, and by occasionally changing the direction of flow, the deposition of sludge is minimized. The suction well is 10 ft. wide and 10 ft. deep, and the diameter of its center line is 52 ft.

The circular pump well is within the annular suction well and is 38 ft. in diameter. The floor is about 44 ft. below that of the motor room, which is slightly above ground level. There are three intermediate floors or balconies, at each of which are steady bearings for the vertical pump shafts.

The pumping equipment consists of six centrifugal pumps, each connected by a shaft to a vertical electric motor on the main operating floor. Three of the pumps have each a rated capacity varying from 5

[1] *Eng. News*, 1916; **75**, 1224.

to 15 million gallons daily against a head of 39 ft.; and are driven by 150-hp., variable-speed, alternating-current motors of the commutating brush-shifting type, with pilot motors for shifting the brushes. At 10 million gallons daily rate, the speed of these pumps is 385 r.p.m. The other three pumps have capacities of 10 million gallons daily each against a head of 39 ft., and are driven by 100-hp., constant-speed, alternating-current motors. These pumps are run at a speed of 385 r.p.m.

The pumps are set 3 ft. below the maximum water level in the suction well, and priming is, therefore, unnecessary.

Each pump has a 30-in. suction and 24-in. discharge nozzle. The discharge pipes are connected in pairs, one of the constant-speed and one of the variable-speed pumps discharging into a 36-in. discharge pipe, while the three 36-in. pipes lead to a 5-ft. force main.

The controller is so designed as to automatically start, speed up, slow down, and stop, the various pumps in any desired order, with not more than five pumps operating at once. Manual operation is also possible if desired, or in case of failure of the automatic apparatus.

Washington.—The sewage pumping station at Washington, D. C., designed by Asa E. Phillips, has been much praised by engineers, European as well as American. The general arrangement of it and of the conduits leading to and by it, which form one of its most interesting features, is shown in Fig. 303.[1] At this station the entire sewage of the city is pumped through a pair of 60-in. pipes about 18,000 ft. long to a point in the Potomac River about 800 ft. from shore. The large conduits on either side of the station discharge into the Anacostia River, on the bank of which the station stands, the storm water from a considerable part of the low-lying portion of the city. A part of this storm water is discharged by gravity, while another part must be pumped at certain stages of the river.

The Tiber Creek and Jersey Ave. high-level intercepting sewer passes along the east side of the pumping station. Before it reaches the station its lower portion has a section 14 ft. wide and 14 ft. 3 in. high, with a cunette, or dry-weather channel, diverted near the station into a 5-ft. circular conduit, into which the east-side intercepter 6¼ ft. in diameter, also discharges. Beyond the point where the dry-weather channel is led to one side, the Tiber Creek sewer continues as a twin section each channel being 12 ft. wide by 10½ ft. high, the invert level with the berm of the cunette section. On the west side of the pumping station the B Street and Jersey Avenue trunk sewer extends. This also has an 18-ft. cunette section 16 ft. high, before it reaches the pumping plant. Where the dry-weather channel, or cunette, is diverted to one side, the main sewer becomes a twin section, each side being 12 ft. wide and 10½ ft.

[1] *Eng. Record*, 1908; **58**, 237.

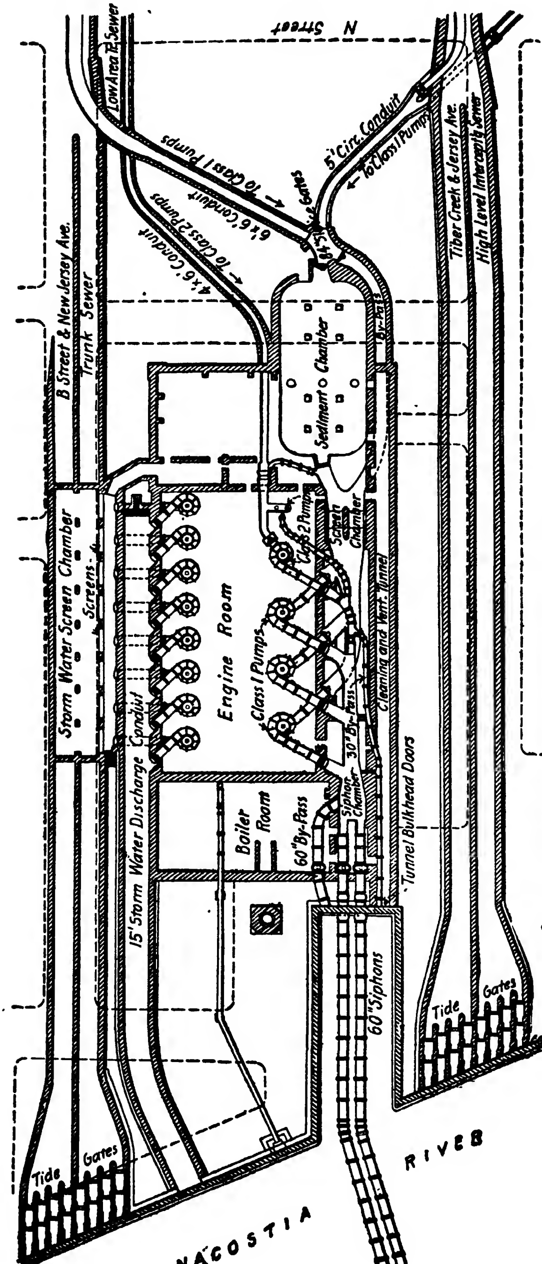


FIG. 303.—Plan of pumping station at Washington, D. C.

high. All these sewers are built of concrete with a lining of vitrified brick on the portion of the invert subject to greatest wear and red brick on the other parts of the invert over which sewage is likely to pass at some time.

The diversion conduit for the dry-weather sewage from the B Street sewer is 6 by 6 ft. in size and joins the 5-ft. circular conduit from the Tiber Creek sewer at a gate chamber containing two 84-in. sluice gates. One of these admits the sewage, during the normal operating conditions, into a sediment chamber 50 by 104 ft. in plan, having a groined arch roof carried by columns 3 ft. square and 16 ft. apart in the clear. This chamber extends partly under the pumping station and is large enough to reduce the rate of flow of the sewage to considerably less than 1 ft. per second. The sediment which is collected in the chamber is removed in ⅔-cu. yd. buckets. These are brought into the chamber on cars run into it on an industrial track laid on the floor, and are filled by hand. The cars are run under a hatch in the roof and the buckets are lifted from them to a trolley on an overhead track at a much higher elevation, by which they are transferred to the river, where their contents are dumped into a barge. The overhead track runs for part of its length through an 8- by 8-ft. passage or tunnel, which is also used as a part of a system of ventilation worked out so completely that no offensive odors have been detected about or in the station.

The sewage is drawn from this chamber into an 8-ft. conduit having a check gate and a twin screening chamber. This screen chamber is 30½ ft. long, 20½ ft. wide, and divided into two equal portions, each with two screens of ¾-in. rods on 2¼-in. centers, operated by hydraulic cylinders and counterweights. The trash from the screens is removed through a branch connection with the conveying and ventilating tunnel just mentioned. The sewage passes through this screening chamber into a suction chamber, from which three centrifugal pumps draw their supply. These lift the sewage into a 16- by 22- by 40-ft. siphon chamber at the head of an inverted siphon under the Anacostia River, which forms the first part of the outfall sewer. The gate valves on the head of the two pipes forming the siphon have their seats on the downstream side cut away so as to leave no bottom slot in the valve bodies in which sediment can be collected. In case the sediment chamber is out of service for cleaning, a bypass delivers the sewage from the gate chamber directly to the pumps. The latter are known as Class I pumps, to distinguish them from two of smaller capacity installed for a special purpose. The sewage from a small low-lying district served by a separate system, independent of the trunk and intercepting sewers, is delivered through a 3½-ft. sewer which has no connection with the settling basin, but runs directly to these smaller pumps known as Class II, an arrangement necessary to obtain proper hydraulic gradients.

The pumps discharge the sewage into the siphon chamber or through an emergency bypass into the river. There is a screen chamber in the suction conduit of these pumps, and a by-pass is provided so that either Class I or Class II pumps can temporarily be used for the service of the other.

The storm water delivered through the Tiber Creek sewer passes directly into Anacostia Creek through the tide gates on the bulkhead, as indicated in Fig. 303. The storm water brought down by the B Street and New Jersey Avenue sewer must pass first, however, into a storm-water chamber, 160 ft. long, 36½ ft. wide and 16 ft. high, having a roof of concrete arches carried by I-beams. Along one side of this chamber are openings fitted with screens of 1½-in. wrought-iron pipe on 4½-in. centers placed on an inclination of 1 to 6. An elevated platform between the walls of this chamber and the pumping station has been constructed for use in cleaning the screens. When the elevation of the water in the river permits, the storm water passes directly through this chamber into the river. When the latter is high, however, tide gates prevent a backflow into the conduit and the storm water that comes down is pumped from the chamber into a 15-ft. discharge conduit at a considerably higher level, eight pumps being provided for this special purpose. It will be observed that it is also possible to utilize the Class I pumps for handling some of this storm water, in case of emergency.

The pumping station has at the land end a three-story 75- by 120-ft. section used for office and shop purposes; in the middle there is a 90- by 170-ft. engine room, and on the river front a 60- by 120-ft. boiler house with elevated coal bunkers. The Class I pumps are three in number, each driven by triple-expansion engines and rated at 100 cu. ft. per second at a head of 27 ft. One of them is a reserve. There are two Class II pumps, one a triple of a capacity of 32 cu. ft. per second raised to a height of 29 ft., and the other a compound of equal capacity. The storm-water pumps discharge under a variable lift; each is capable of raising 100 cu. ft. per second to a maximum height of 15 ft., but they are particularly effective at their usual lift of 3 to 8 ft. Owing to the fact that they are in operation only a portion of the time, they are driven by compound engines. All engines but one are of the horizontal type, without fly-wheel, direct-connected to a vertical pump shaft, first developed by the Allis-Chalmers Company for one of the Boston sewage pumping stations. The Washington pump setting differs from that of earlier stations in the omission of separate chambers for each pump, for in Washington the entire basement of the engine room serves as a large dry well. The only vertical engine is the compound driving one of the Class II pumps, a unit which had been used during the con-

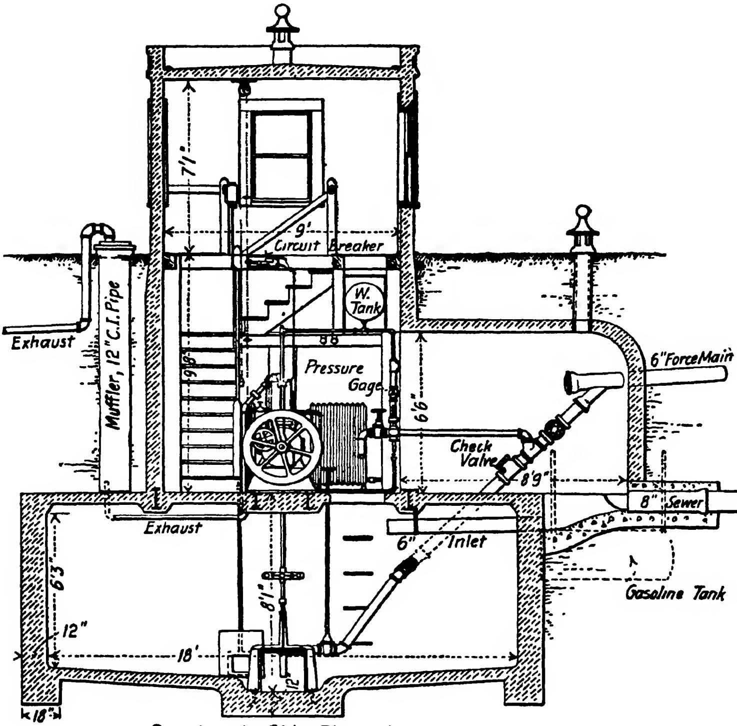
struction of the station, and was in good enough condition to be installed as a reserve in the permanent plant.

The engines are supplied with steam by six water-tube boilers, each of 275 hp., with automatic stokers, fuel economizer, complete mechanical coal-handling machinery, and the other accessories and auxiliaries of a high-grade power plant.

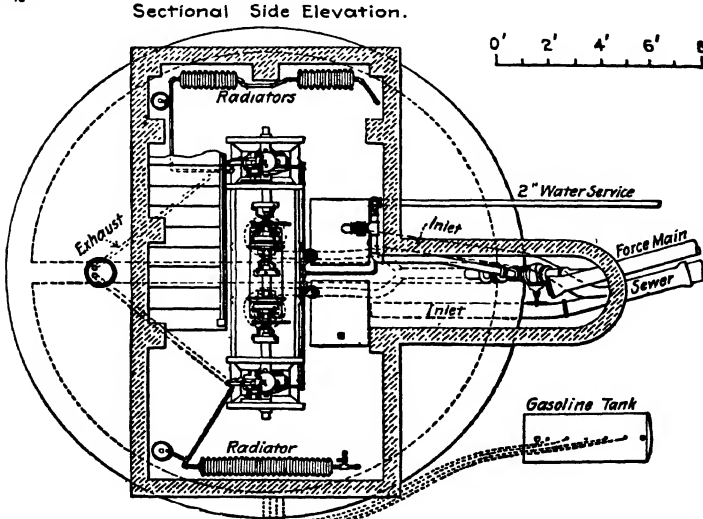
Austin, Texas.—The sewage treatment plant includes a pumping station built in 1918 from plans of John H. Gregory. Sewage flows through a 30-in. sewer to a suction well 8 by 22 ft. in plan (interior dimensions). A variation in water level of 4 ft. is provided for, high water being 2 in. below the center of the 30-in. sewer. The pumps are two 10-in. horizontal centrifugal pumps, directly connected to electric motors; space is left for a third pump. The center of the pumps is 2 ft. 4 in. below high water in the pump well, so that priming is unnecessary. The maximum lift is about 30 ft. The force main is 24 in.; after passing through the wall of the building it makes a turn of 180 deg. and returns through the station at a higher level, thus allowing the Venturi meter to be placed within the station without increasing the size of the structures.

The pumping station proper (excluding the suction well) is 19 by 28 ft. in interior dimensions. The ground floor contains an office and laboratory, and the chlorine apparatus; the pump floor is 21 ft. lower and contains the pumps, motors, and valves. The Venturi meter and the switchboard are in a gallery at an intermediate elevation.

Marblehead, Mass.—The pumping station built in 1927 at Marblehead, from plans of Frank A. Barbour, may be taken as typical of good current practice. The town is situated on the coast and had a population of 8,214 in 1925. Sewage flows by gravity to a pump well forming part of the pumping station, and is pumped through a 10-in. force main to an ocean outfall. The pump well is 10 by 14 ft. in plan, and has a storage capacity of 3,000 gal. between the elevation of the invert of the 15-in. sewer and the low-water elevation, at which the pumps are stopped. If water should rise appreciably higher than the sewer invert, an overflow from a manhole outside the station would come into action if the stage of the tide permitted; at very high tides the water might rise 8 ft. higher before the overflow could come into action, corresponding to a further storage of 8,000 gal. There is a bar rack of the type shown in Fig. 290, in front of the inlet sewer, composed of ¾-in. square bars spaced 1¾ in. on centers. Sewage falls through the rack into the trough across the end of the pump well. This trough is ordinarily drained by two 2-in. bleeders, but if these should become clogged, sewage will overflow the lip of the trough. The two 3-in. vertical centrifugal pumps are located in a dry well, also 10 by 14 ft. in plan. Each has a gate valve on the suction, and a gate and a check valve on

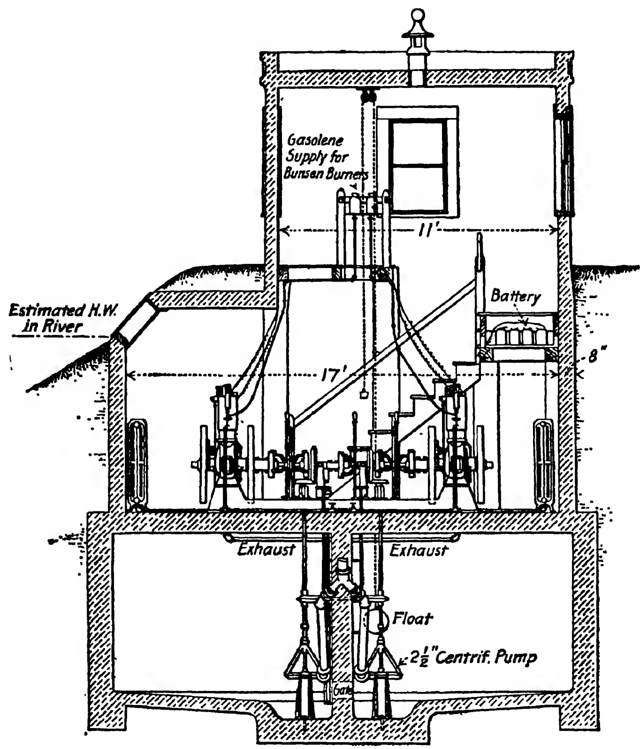


Sectional Side Elevation.



Sectional Plan.

the discharge. The superstructure is built over the dry well, with its floor 16.25 ft. above that of the latter, and contains the electric motors and switchboard. There is a 12-in. float pipe for each pump, so that each pump may be separately controlled automatically. Provision is made for ventilation through a screened air inlet near the floor, and louvres in the gable ends just below the roof.



Sectional Front Elevation.

FIG. 305.—Pumping station at Newton, Mass., as originally built.

Newton, Mass. (Figs. 304 and 305).—A pumping station built for temporary service at Newton, Mass., from the plans of the late Irving T. Farnham, city engineer, illustrates a type of plant where the water end must be at a low elevation and internal combustion motors are desired for operation. It was constructed in 1903 as an alternative to a very expensive sewer for the small number of people to be served until the district was developed considerably beyond its population at that time,

but was still in use in 1928. The sewage is delivered to a circular tank 18 ft. in diameter and about 7 ft. deep inside, holding about 13,000 gal. The walls are 12 in. thick, on 18-in. footings, and the bottom is 6 in. thick with a downward slope to a central sump about 1 ft. deep. The tank is divided by a 10-in. wall through the center into two halves, and an 8- by 8-in. sluice gate at the bottom of the wall enables either side to be shut off for repairs or cleaning. The tank has a 5-in. roof about 10 ft. below the surface of the ground. This is constructed of reinforced concrete and carried by I-beams.

The pump house is 9 by 17 ft. in plan, 9 ft. 8 in. deep underground, and 7 ft. 1 in. high above ground. The basement walls are 8 in. thick, of reinforced concrete, and are strengthened by a number of buttresses; an extension of this basement, 8 ft. long and 3 ft. wide, serves as a valve chamber and has independent ventilation. The superstructure has walls 6 in. thick with a roof 7 in. thick at the ridge and 5 in. at the sides.

The original pumping plant consisted of two 2½-in. vertical centrifugal pumps operated by two 6-hp. gasoline engines. The connections were such that either engine could be used to drive either pump, or both engines used to operate either or both pumps. The pumps were started by an attendant in a water-works pumping plant 300 ft. distant but were stopped automatically by means of a float-operated device, which cut off the electric ignition. The plant had a total capacity, on test, of 400 gal. per minute. The total lift was 30.7 ft.

The present pumping plant (1928) includes one of the original 2½-in. centrifugal pumps driven by a 6-hp. gasoline engine, and having a capacity of 200 gal. per minute against a head of 30.7 ft.; one 4-in. centrifugal pump driven by 10-hp. electric motor, having a capacity of 300 gal. per minute against a head of 50 ft., installed in 1917; and one 3½-in. centrifugal pump driven by 20-hp. electric motor, having a capacity of 450 gal. per minute against a head of 65 ft., installed in 1924. The quantity of sewage pumped in 1926 was 22,400,000 gal., and the direct cost of power (*i.e.*, cost of gasoline and of electric current) amounted to $830.

The construction cost of the original plant in 1903 was $6,700; of the pump and motor added in 1917, $2,040; and of that installed in 1924, $2,913; making a total cost of $11,653, without deduction for the gasoline engine and pump removed, and without adjustment for changing scale of prices.

Ejector Pumping Stations at Larchmont, N. Y.—Four pneumatic ejector stations were built at Larchmont in 1925 from plans of Nicholas S. Hill, Jr. The two largest are equipped to lift 212 and 170 gal. per minute against heads of 34 and 22 ft., respectively. Each of them is built below ground, in a park or residential district. The floor of the ejector room is 4 or 5 ft. below the elevation of the influent sewer. The ejector room contains two Jennings ejectors of suitable capacity, and

the compressor room above it contains two Nash "Hytor" air compressors, with motors and switchboard. A sump pump is also provided, and in the case of the station which is entirely below ground, a small blower for ventilation. These stations are from 9 by 12 ft. to 12 by 16 ft. in plan, and from 17 to 22 ft. in total height.

INDEX

D

E

H

I

J

M

N

O

P

Q

R

S

Y

Z

487